EVALUATION AND IMPROVEMENT OF INTAKE STRUCTURE POSITION FOR OVERALL OPTIMIZATION OF HYDROPOWER PRODUCTION, A CASE STUDY OF GRAND ETHIOPIAN RENAISSANCE DAM

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June 2018

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BY

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A Thesis Submitted to the Collage of Civil and Architecture Engineering for the Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering

(Hydraulic Engineering)

ADDIS ABABA SCIENCE AND TECHNOLOGY UNIVERSITY

June 2018

Declaration

The present study ‘‘Evaluation and improvement of intake structure position for overall optimization of hydropower production, a case study of grand Ethiopian renaissance dam’’ is a thesis submitted to the Addis Ababa science and technology university, collage of civil and architecture engineering, in partial fulfilment of the requirements for Master of Science Degree in hydraulic engineering.

The work presented in this thesis is my own and all significant outside inputs have been identified and acknowledged to the best of my knowledge. It is intended that this study will be updated when more data becomes available.

Tamrat Zewdie

Certificate

Addis Ababa Science and Technology University Post Graduate Studies

This is to certify that the thesis prepared by **Tamrat zewdie**: evaluation and improvement of intake structure position for overall optimization of hydropower production, a case study of grand Ethiopia renaissance dam. Submitted in partial fulfillment of requirements for the degree of masters of Science in Civil Engineering (Hydraulic Engineering) complies with the regulations of the university and meets the standards with respect to originality and quality.

Signed by examining committee:

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Abstract

Ethiopia's different hydraulic structure projects are widely under construction and others under planning stage to reduce the boost the economic growth of the country and meet middle income transformation plan of the country. In line with that, hydropower and irrigation projects are among the major civil structures under construction in the country. Of these hydropower plants Grand Ethiopia Renaissance Dam project is a great role in our country today and in feature. The dam is located in Benshangul Gumuz Regional State. The main objectives of this study to evaluate the current intake position +587.85 masl and the previous intake position +560 masl with regards to vortex formation, economy, sediment and debris and effective use of water and by taking the result that gates from the two intake structure position evaluation and comparing each other. The two intake position that have around 18.85 meter difference this difference position has its own advantage and dis advantage based on this the new intake position has advantages on the oldest intake position on economy and sediment and debris, on other hand the oldest takes advantage on the current position on vortex formation and water usage this indicate the intake position can be reanalyze and select the best intake position.

Key words: intake position, vortex, sediment, reservoir, cost.

Acknowledgement

Honor for his intimacy, GOD who ultimately enabled me from the very beginning of
My studies up to the end page of this report.

I wish to express my sincerest thanks to those who have helped in
Executing this work. This includes my family, colleagues and friends.

My profound gratitude is expressed to my advisor: Dr. Mesay Daniel
Special thanks for guidance, supervision, support and advice.

Thanks also go to the staff of the METEC Research and Development Center specially energy institute

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MW…………………………………………..……mega watt

NGOs…………………………………………...non-governmental organizations

GTP…………………………………………….growth and transformation plan

GERD…………………………………………..grand Ethiopia renaissance dam

HPP……………………………………………..hydro power plant

EM………………………………………………elector mechanical

METEC…………………………………………metals and engineering corporation

EEPCO…………………………………………Ethiopia electricity power corporation

Q………………………………………………..discharge

TR ……………………………………………..return period

PMF…………………………………………….probable maximum flood

GWH……………………………………………gaga watt hour

FR……………………………………………….fraud number

G ………………………………………………..gravity

M………………………………………………..meter

V ………………………………………………..velocity

S ……………………………………………….sub mergence height

D………………………………………………..diameter

DWL…………………………………………….dead water level

USBR…………………………………………… United state bureau reclamation

Z ………………………………………………...elevation

A…………………………………………………area

NWL……………………………………………….normal water level

NOL………………………………………………..normal operating level

FSL…………………………………………………full storage level

MOL…………………………………………….....minimum operating level

M tons/year…………………………………………Million tons per year

U/S…………………………………………………up stream

KG………………………………………………….kilogram

Mm^3…………………………………………………million meter cube

Mm^3…………………………………………………million meter square

Masl…………………………………………………meter above sea level

Bm^3…………………………………………………billion meter cube

H……………………………………………………depth

Cc……………………………………………………coefficient of contraction

M/s………………………………………………….. Meter per second

IHA…………………………………………………international hydropower association

F………………………………………………………catchment area

1 Introduction

1.1 Background

Currently, in Ethiopia different water development projects are under construction widely and others are planned to bring the country's economy to middle class. Storage type hydropower development and large scale irrigation projects are the two major water resource development related activities. However, the hydropower potential of the country is estimated as according to Ethiopian electric power corporation 45,000 MW, but only developed 4,054MW and increase with rate of 0.7% of the potential is used until 2017.

To balance demand and supply of power of the country in the future the country planned to construct Grand Ethiopia Renascence Dam, Gibe III and GenaleDawa 3 were included in Ethiopia's 2010–15 development strategy, the first Growth and Transformation Plan (GTP1). Together, they are set to boost the country's installed capacity to over 10,000 MW by late 2016. Within the GTP2 period, Ethiopia plans to commission a further 3,900 MW of new hydropower, including the 254 MW GenaleDawa 3 in 2016, and Geba I and II 385 MW in 2018. Further developments include Gibe IV and Gibe V (2,000 MW and 600 MW, respectively), as well as the Upper Dabus (326 MW) and HaleleWerabesa (436 MW). Ethiopia also plans to begin construction on a further 7,500 MW spread across ten projects by 2020 and in late 2030 total installed capacity 22,000 MW total (IHA et al, 2017).

From under construction hydropower projects the GERD is the largest hydropower plant dam in Africa with 75 billion meter cube reservoir capacity, 6000MW energy production, 1780m dam length, 145m dam height while its saddle dam is 4.8km length with 45m height. This project covers about 15% of the total estimated hydropower potential of the country. Because of the political case against the downstream beneficiary's countries, the project proposed for only hydropower generation as non-consumptive purpose rather than use for consumptive purpose like agricultural production (irrigation) as multi-purpose. But in case of GERDP, according to the political guideline what percent respected from the upstream users for the downstream users is under question because the project system is totally non-consumptive. Even to use effectively for hydropower by adjusting operation schedule with respect time we have within a year by what percent the project production to be maximized is also under question. So to have a maximum output from any project within political guideline standard over any natural resource, more studying and analyzing from time to time is very important. According Ukhydro Project Public Company carried out optimization of the HPP in 2011. The design of GERD has an installed capacity of 6000 MW with 16 hydro-units of 375 MW. In order to improve the Hydropower projects technical and economic characteristics, which was approved and became the basis for further designing? In accordance with GERD project optimization the installed capacity was developed to 6450 MW with installation of 14hydro-units of 403.125 MW and 2 early generation units of 403.125MW at which 10 turbines are located in the right-ban k powerhouse and 6 in the left-bank one. (EEPCO et al, 2013)

1.2 Problem statement

Hydropower dam is used to store water to be used in dry period power production. There are different structural parts in a hydropower plant starting from reservoir up to tail race that use to produce power. One of the main structures is the intake structure that interfaces between the reservoir and the turbine. The intake design and position influences the quantity quality of water withdrawn from the reservoir (sediment-laden, debris-laden, fishery impact), and, formation of vortex. The intake level of GERD has been changed recently. Such new intake level may under-estimate the potential benefits of getting from high head of the project. The changed intake structure position raised was the potential threat of the project in increasing the dead water storage. The changed intake structure position has a direct impact on the potential power production capability of the project and life span of the project. This study will try to evaluate the intake position of the dam from technical point of view. Technical advantage here means maximizing the total energy production and life span of the project. This paper will evaluate and improve the intake structure position and estimating the amount of water not accounted under the productive because of different case like changing of intake position and its outcomes.

1.3 Objective

General objective

To analyze and evaluate the current intake position and suggest an optimal position with respect to overall maximization of the pant

Specific objectives

- To identify the main criteria to fix the intake structure position
- To identify the technical advantage or disadvantage of the current intake structure position.
- To estimate the amount of water volume lost in case of changing the position of intake structure
- To propose a new intake structure position, that would maximize the technical benefits of the project,

1.4 Scope of study

The study has been done by focusing on the evaluation of the already proposed intake position and with improving intake positon to optimize the total output and to minimize the risk of vortex formation on the GERDP in fixing the new intake position.

1.5 Layout of thesis

The thesis is composed of five chapters namely, chapter one, which is the introduction, stating the case study project, objectives and statement of the problems. Literature review and previous study of Blue Nile River (GERD) are found in chapter two. In chapter three, the background about study is presented and the main data and information available about the GERDP and describes the methods used to evaluate intake structure position. Chapter four shows the analysis of the data collected and discussion of the results. Chapter five gives the conclusion and recommendations based on the discussion of the results as well as some recommendations for future works, References used and Appendix are listed at the end of the Thesis.

2 Literature Review

2.1 General

The main function of an intake structure is to divert water into the waterway, which conveys water flow to the power plant in a controlled manner. Intake structures are categorized as Structures that take water directly from the water flow and divert it to the penstock, Intakes which divert the flow through an auxiliary structure, Structures located in reservoirs, e.g., towers.

The main components of an intake are Screen to prevent floating material from entering the waterway, Screen cleaning system, usually a crane that removes debris from the screen, Intake gate to physically disconnect the water flow in case of maintenance, penstocks, tunnels, and surge tanks(canyon hydro et al,2002)

2.2 Hydropower potential in Ethiopia

Ethiopia is fast becoming a center of industry and new infrastructure, with the aspiration to achieve middle- income status by 2025. Since 2011, Ethiopia has implemented the Climate- Resilient Green Economy (CRGE) strategy, which substitutes conventional development by means of harnessing clean energy sources like hydropower, wind, geothermal, solar and biomass, and implementing energy-efficient technologies in the transport and industrial sectors. With its 2010 Growth and Transformation Plan I (GTP-I), Ethiopia aimed to quadruple installed capacity by prioritizing large hydro developments and achieving total power installed capacity of 10,000 MW by 2015. The government published the GTP-II for 2016-20, with the objective of reaching total installed capacity over 17,208 MW. Hydropower is set to make up about 90 per cent of the power supply. Ethiopia has some of the richest water resources in Africa, distributed across eight major basins with an exploitable hydropower potential of 45,000 MW. Over half of this potential is located in the Abbay and Omo river basins, where the nearly-completed Grand Ethiopian Renaissance Dam (GERD) and the recently-completed 1,870 MW Gibe III project, are located. (IHA et al, 2017)

Gibe III, the tallest roller-compacted concrete (RCC) dam in the world, with 246 m dam height and 630 m crest length, was inaugurated in December 2016. The USD 1.8 billion construction was financed 40 per cent by the Ethiopian Government, and 60 per cent by the China Exim Bank. While all turbines have been installed and commissioned, not all are yet online, as reservoir filling is still in progress.

GERD's construction is progressing according to the timeframe, with more than half already complete. Sudan, Egypt and Ethiopia agreed a new declaration for cooperation in March 2016 that alters the 1929 treaty, where Egypt had a veto over any upstream projects in the Nile River. In addition to GERD's construction, the 254 MW Genale Dawa hydropower plant is near completion. Salini Impregilo, the constructor of Gibe III and GERD, signed a USD 2.8 billion contract with EEP in March 2016 to build the 2,160 MW Koysha hydro project. The project is financed between EEP and the Italian Export Credit Agency. Downstream from Gibe III, this is the fourth plant in the Omo River cascade scheme, which envisions a fifth dam further downstream. Other hydropower projects in the bidding phase are Tams (1,700 MW), Chemoga Yeda (280 MW), and the Geba complex (385 MW). (IHA et al, 2017)

PowerChina Huadong Engineering Corporation completed the rehabilitation – at a cost of USD 14 million – of Ethiopia's oldest hydropower plant, the 6.6 MW Aba Samuel, which dates back to 1941.The GTP-II also envisions exploiting alternative sources such as wind, solar, geothermal and biogas resources. The exploitable capacity from other sources is estimated at 1.3 million MW (wind) and 7,000 MW (geothermal). The 1,000 MW Corbetti geothermal power project, with a cost of USD 4 billion, is expected to be commissioned in 2018. Currently, the 300 MW Aysha, 100 MW Debreberhan and 150 MW Itaya wind farms are under development, with others like the 100 MW Assela under study. (IHA et al, 2017)

Ethiopia is also rapidly expanding its transmission and distribution network in order to light up the country. Existing cross-border interconnections include 100 MW to Sudan and 50 MW to Djibouti, while the 1,000 km Eastern Electricity Highway Project (500 kV) will be capable of exporting 2,000 MW to Kenya upon completion in 2018. The country has ambitions of becoming the 'energy hub' within the Eastern Africa Power Pool (IHA et al, 2017).

2.3 Review on the case study project (GERDP)

The annual average inflow rate, rain fall, runoff and potential vapor-transpiration are 1547m^3/s, 229mm, 765mm, and 1080mm respectively and about 10% is loss of the total storage volume per 50 years by entering sediment to the GERDP reservoir (EEPCO level I, volume I, 1559-1-t15, 2013). On all the general design of the project, different analysis of reservoir operation like elevation-duration-curve, volume-duration-curve, area-volume-curve, and elevation-volume-curve etc. were done during the basic design (EEPCO level I, volume I, 1559-1-t15, 2013). The criteria that stipulate the passage from one flow condition to another were defined by experimental tests carried out on tunnels having a circular profile. The selected flows have been considered for the analysis of tail water levels during operation period of the Plant: Q=1540 m^3/s corresponding to Average Runoff; Q = 3000 m^3/s corresponding to Normal HPP Discharge; Q = 4700 m^3/s corresponding to Maximum Discharge; Q=22800 m^3/s corresponding to spilled flow for an Exceptional Flood Tr =10000yrs; Q=27000 m^3/s corresponding to spilled flow for an Extreme Flood PMF. The reservoir of GERDP HPP carries out long-term flow regulation. Its live storage was 59.5 km^3, which is 20% more than average annual river flow at the HPP site (49.2 km^3), is Great potential for flow regulation allows for redistribution of power generation from high-water years, seasons, months to low-water ones, which ensures HPP operation in any mode required by consumers. The reservoir and HPP commissioning will have a positive impact on energy-conversion efficiency of downstream HPPs: power generation will increase due to reduction of spilling; firm power will grow due to the increase of regulated discharges during drought seasons (EEPCO et al, 2013).

In order to determine parameters of energy-conversion efficiency of the HPP with installed capacity of 6,000 MW, water power calculations were performed using a calendar method by applying during a series of 95-year average monthly discharges from 1911 to 2005. Average annual water discharge over this period is 1559 m^3/s. Volume of annual flow varies from 20.69 km^3 to 69.84 km^3, with average annual volume 49.2 km^3, annual power generation with average annual flow is equal to 16204GWh at average annual capacity 1848 MW, the reservoir simulation model was implemented using the sequential stream flow routing (SSR) method. (EEPCO et al, 2013)

The HPP GERDP installed capacity was developed to 6000 MW with installation of 16 hydro-units of 375 MW, including those 10 in the right-bank powerhouse, and 6 in the left-bank one, power production was increased by about 2.5% and the costs were reduced in comparison with the Basic Design, 2010, at the water level in the reservoir is between elevations 637.0 and 590.0 m, the HPP operates at firm flow 1285 m^3/s. (EEPCO et al, 2013)

2.4 Water intake types

The first thing for the designer to do is to decide what kind of intake the scheme needs. Notwithstanding the large variety of existing intakes, these can be classified according to the following criteria: the intake supplies water directly to the turbine via a penstock This is what is known as power intake or forebay, The intake supplies water to other waterways-power canal, flume, tunnel, etc.- that usually end in a power intake This is known as a conveyance intake The scheme doesn't have any conventional intake, but make use of other devices, like siphon intakes or French intakes that will be described later. In multipurpose reservoirs built for irrigation, drinking water abstraction, flood regulation, etc. the water can be withdrawn through towers with multiple level ports, permitting selective withdrawal from the reservoir's vertical strata or through bottom outlets (canyon hydro et al, 2002)

The siphon intake renders intake gates unnecessary, and the inlet valves (provided each unit has its own conduit) may also be eliminated, reducing the total cost by 25-40 percent, and reducing the silt intake. The water flow to the turbine can be shut off more quickly than in a gated intake, which is beneficial in a runaway condition. A siphon intake built on an existing dam, with very small civil works. The siphon can be made of steel, or alternatively in countries where the procurement of fabricated steel is difficult, in reinforced concrete, with the critical sections lined in steel. The French or drop intake is essentially a canal built in the stream-bed, stretching across it and covered by a trashrack with a slope greater than the streambed slope. The trashrack bars are oriented in the direction of the stream flow. A drop intake installed in a mountain stream in Asturias (Spain). In France EDF has improved this type of intake, placing the bars as cantilevers to avoid the accumulation of small stones commonly entrained by the water. The Coanda type screen is an advanced concept of the drop intake, incorporating the Coanda effect, well known in the ore separation industry, to separate fish and debris from clean water. Essentially it consists of a weir with a downward sloping profiled surface of stainless steel wire screen mesh on the downstream side and a flow collection channel below the mesh ñ as in the drop intake. The mesh wires are held horizontal ñ unlike the drop intake- and are of triangular section to provide an expanding water passage. Water drops through the mesh with debris and fish carried off the base of the screen. The screen is capable of removing 90% of the solids as small as 0.5 mm, so a silt basin and sediment ejection system can be omitted. (Canyon hydro et al, 2002)

2.5 Components of Intake structure

Intake structure has different components that use for different purpose the main components are trash rack, trash rack supporting structure, stop log and control gate, anti-vortex arrangement, bell mouth & transition. (Blaisdell, F. W et al, 1982)

2.5.1 Trash rack

Trash rack is a screen provided at the intake to prevent entry of floating debris like grass, leaves, trees, timbers etc., in to water conductor system. Each screen consists of vertical trash bars welded space bars consisting of flat/ channel sections. The screens are assembled in small panels for easy handling for maintenance. The trash bars are generally of mild steel flats with rounded edges at both upstream and downstream for smooth flow. The spacing of trash bars depend up on the type of turbine, its dimension and peripheral speed of the runner. Following criteria should kept I mind while designing trashracks. Trash bars should be so spaced that the net opening between them should be at least 5 mm less than the minimum opening between turbine runner blades, The spacing of the bars should be adjusted so that the ratio of forcing frequency to natural frequency of bar is less than 0.6., The trash rack should also be designed to withstand the effect of submerged jets in the case of pumped storage scheme, The design loads for trash racks are the dead weight of the assembly, the water pressure and dynamic pressure of the floating materials. An unbalanced pressure is also developed on account of partial or total clogging of the racks. Emil Mosonyi suggests a differential head of 1 to 2 m under normal conditions and 4 to 5 m under exceptional conditions.

U.S.B.R. recommendations are that the racks are to be designed to fail at 12 m differential hydraulic head for deeply submerged intakes and where submergence is 6 m or less, the head is to be taken as 2;3rd of the maximum depth of submergence. Ewe practice is to consider 6 m of differential head in design to take care of dynamic pressure of floating material and unbalanced pressure due to partial clogging of the racks.50% clogging is assumed in determining hydraulic pressure acting on racks, The velocity of flow in front of the screen has to be of such a value as to minimize the loss of head. Further, higher velocity may cause vibration in trash rack structure and may lead to its failure. The velocity of flow through the rack may be about 0.75 m/sec, if manual cleaning is resorted to and1.5 m/sec, if the cleaning is by racking machines. Trash racks are to be cleaned frequently. For small stations with depth of racks 4 to 5 m, and where the floating material is small, manual cleaning is possible. If the floating material is large and height of trash rack structure is more, mechanical cleaning machines should be deployed for cleaning and Trash racks should also be inspected at least once in a year to timely detect any damage to the racks. (Canyon hydro et al, 2002)

2.5.2 Trash rack supporting structure

This is a reinforced concrete structure of columns (piers) and beams (ribs) on which the trash rack screens rest. The structure may be vertical or inclined with respect to the axis of the penstock joining the intake. The Following criteria I guidelines should be kept in mind while designing trash racks supporting structures; The design of the supporting structure is made while considering the loads transferred by the trash rack, dead load of structure, dead and live load of the operating platform I top slab. A differential water head of three to six meters is considered depending upon the cleaning efficiency of trash racks. The columns and beams in the flow direction are shaped to enhance smooth flow, the shape of trash rack structure may be adopted to meet the requirements of the layout of head works layout and head loss. For instance, for high dams with nearly vertical upstream face, semi-circular trash rack structure is usually preferred to provide the required trash rack area economically. For low dams or diversion structures, a straight trash rack is usually preferred. Care should be taken to prevent dead zones of water and uneven or irregular flow patterns in the tunnel, formation of dimples, dye core and air core vortices, water circulation and other flow irregularities during operation in pumping, turbine or combined modes under symmetrical and asymmetrical operation of unit., No part of the trash rack structure should fall within 80 per cent of the intake height, he, from the center point of intake., For an uptight semicircular intake structure, the racks should be located on a semicircle in plan with a minimum radius of 1.1428 be, where, be is the width of opening. For an inclined semicircular intake structure, the racks should be located on a semicircle or a plane perpendicular to the axis of the structure and satisfying the other criteria as for the upright structure. In plan the racks would be laid out on an ellipse, the semi• major axis of which should have a minimum value of 1.1428be/Cosq, where q is the inclination of the trash rack axis to the vertical. The semi minor axis of the structure is parallel to the dam face and would have a value of 1.1428be. The trash rack screens should be inclined in a three dimensional plane with a bottom corner of the tower screens resting over the base footing.& The approach apron should not be placed closer than 30 percent of the intake height he from the lower edge of the intake orifice. (Canyon hydro et al, 2002)

2.5.3 Stop logs and control gates

Stop logs and control gates are provided for regulation of flow into the water conductor system. Stop logs are used when the intake gate needs maintenance and repairs. Grooves for stop logs and gates are provided generally in the intake body or piers. Following criteria I guidelines should be kept in mind while designing Stop logs and control gates; The operating platforms of stop log and gates are kept at such a level that the equipment's are approachable for operation under all conditions. , The control gate may be installed at the entrance or after the bell mouth section. In the former type, the gate may be operated from the top of the dam and in the latter case, generally, it is operated through a shaft or gate gallery provided in the body of the dam. , An air vent downstream of intake gate should be provided to release air pockets trapped along the inflow water. The air vent should be so designed as to admit air at the rate the turbine is discharging water under full gate conditions. (Canyon hydro et al, 2002)

2.5.4 Anti-vortex arrangements

Requirement of submergence depth is primarily an anti-vortex arrangement. Where the required submergence depth is not possible or vorticity is anticipated, additional arrangement is done to prevent formation of vortex at the intake. They may consist of reinforced concrete vertical fins constructed parallel to each other, Dinorwic louvered type or perforated breast walls. The details of these arrangements are finalized through model studies. For the design of perforated breast wall, anti-vortex louvers and vertical fins, one meter differential head may be adopted. (Canyon hydro et al, 2002)

2.5.5 Bell Mouth and transitions

The entrance is shaped in the form of a bell mouth so as to have a smooth flow and reduce losses. As already mentioned, the intake may be inclined or kept vertical with respect to the dam axis.The shape of inlet should be such so as to ensure uniform acceleration of flow. The entrances of penstock and conduits entrances are designed to produce an acceleration similar to that found in a jet issuing from a sharp edged orifice. The surfaces are formed to natural contraction curve and the penstock or conduit is assumed to be of the size of the orifice jet at its maximum contraction. The normal contraction of 40 percent (coefficient of contraction Cc= 0.6) is to be used in high and medium head installations, 30 percent (Cc=0.7) for low head installations and 50 percent for (Cc= 0.5) for re-entrant type intake. The opening area at the inlet= (Penstock area I Cc x Cos f), where, f = angle of inclination of penstock center line to horizontal. Gates need rectangular section for efficient operation and pressure pipe or penstock need circular section for its hydraulically efficient design. Hence, transitions from rectangular section to a circular section conduit is needed for achieving both the objectives. Sometimes transition is also required, when the cross-sectional area of flow decreases or increases due to bifurcation or merger. The transition should be designed in accordance with the following requirements: Transition or turns should be made about the center line of mass flow and should be gradual, The cross-sectional area throughout transition from rectangular to circular section and vice-versa should remain the same so as not to cause any acceleration or deceleration of flow., Side walls should not expand at a rate greater than 5° from the center line of mass flow, All slots or other necessary departures from the neat outline should normally be outside the transition zone. (Canyon hydro et al, 2002)

The geometry of the approach to the power intake should be such that it can ensure economy and better hydraulic uniform flow condition. The flow lines should be parallel, having no return flow zone and having no stagnation. Velocity distribution in front of penstock should be uniform and there should not be any formation of vortices. Formation of vortices at the intake depends on a number of factors such as approach geometry, flow conditions, velocity at the intake, geometrical features of trash rack structure, relative submergence depth and withdrawal Froude number(Fr), etc.To prevent vortices, the center line of intake should be so located as to ensure following submergence requirements, which has been developed by an evaluation of minimum design submergence at prototypes operating satisfactorily. For large size intakes at power plants (Fr= v I bgD < 1/3), especially at pumped storage system, a submergence depth, h = 1 to 1.5 times the intake height or diameter is recommended for medium and small size installations (Fr>1/3), especially at pump sumps, submergence requirements may be calculated using the formula ID= 0.5 + 2 F r. This recommendation is valid for intakes with proper approach flow conditions. With well controlled approach flow conditions, with a suitable dimensioning and location of the intake relative to its surroundings and with use of anti -vortex devices, submergence requirements may be reduced below the limits recommended above. However, recourse to hydraulic model studies may be taken to determine more accurate value depending On other specific parameters of the particular structure. (Canyon hydro et al, 2002)

2.6 Power intake

The power intake is a variant of the conventional intake, usually located at the end of a power canal, although sometimes it can replace it. Because it has to supply water to a pressure conduit ñ the penstock- its hydraulic requirements are more stringent that those of a conveyance intake. In small hydropower schemes, even in high head ones, water intakes are horizontal, followed by a curve to an inclined or vertical penstock. The design depends on whether the horizontal intake is a component of a high head or a low head scheme. In low head schemes a good hydraulic design often more costly than a less efficient one- makes sense, because the head loss through the intake is comparatively large related to the gross head. In high head schemes, the value of the energy lost in the intake will be small relatively to the total head and the cost of increasing the intake size to provide a lower intake velocity and a better profile may not be justified. In a power intake several components need consideration: Approach walls to the trashrack designed to minimize flow separation and head losses, Transition from rectangular cross section to a circular one to meet the entrance to the penstock Piers to support mechanical equipment including trashracks, and service gates, Guide vanes to distribute flow uniformly Vortex suppression devices The velocity profile decisively influences the trashrack efficiency. The velocity along the intake may vary, from 0.8 - 1.0 m/sec through the trashrack to 3 - 5 m/ sec in the penstock. A good profile will achieve a uniform acceleration of the flow, minimizing head losses. A sudden acceleration or deceleration of the flow generates additional turbulence with flow separation and increases the head losses. Unfortunately a constant acceleration with low head losses requires a complex and lengthy intake, which is expensive. A trade-off between cost and efficiency should be achieved. The maximum acceptable velocity dictates the penstock diameter; a reasonable velocity of the flow approaching the trash rack provides the dimensions of the rectangular section. The research department of Energy, Mines and Resources of Canada 10 commissioned a study of entrance loss coefficients for small, low-head intake structures to establishing guide lines for selecting optimum intakes geometries. The results showed that economic benefits increase with progressively smoother intake geometrics having multiplane roof transition planes prepared from flat formwork. In addition, it was found that cost savings from shorter and more compact intakes were significantly higher than the corresponding disbenefits from increased head losses. The analyses of cost/benefits recommend the design of a compact intake it appeared that the length of the intake was unlikely to be the major factor contributing to the overall loss coefficient- with a sloping roof and converging walls. The K coefficient of this transition profile was 0.19. The head loss (m) in the intake is given by Dh = 0.19 v2/2g where v is the velocity in the penstock (m/sec). (Canyon hydro et al, 2002)

A well-designed intake should not only minimize head losses but also preclude vorticity. Vorticity should be avoided because it interferes with the good performance of turbines especially bulb and pit turbines. Vortices may effectively: Produce non-uniform flow conditions Introduce air into the flow, with unfavorable results on the turbines: vibration, cavitation, unbalanced loads, etc. Increase head losses and decrease efficiency Draw trash into the intake the criteria to avoid vorticity are not well defined, and there is not a single formula that adequately takes into consideration the possible factors affecting it.

According to the ASCE Committee on Hydropower Intakes, disturbances, which introduce on-uniform velocity, can initiate vortices. These include:- Asymmetrical approach conditions Inadequate submergence Flow separation and eddy formation Approach velocities greater than 0.65 m/sec Abrupt changes in flow direction Lack of sufficient submergence and asymmetrical approach seem to be the commonest causes of vortex formation. An asymmetric approach is more prone to vortex formation than a symmetrical one. Providing the inlet to the penstock is deep enough, and the flow undisturbed vortex formation is unlikely. According to Gulliver, Rindels and Liblom (1986) of St. Anthony Falls hydraulic laboratories, vortices need not be expected provided. (Canyon hydro et al, 2002)

2.7 Location for the intake and powerhouse

The location of the intake depends on a number of factors, such as submergence, geotechnical conditions, and environmental considerations especially those related to fish life- sediment exclusion and ice formation where necessary. The orientation of the intake entrance to the flow is a crucial factor in minimizing debris accumulation on the trashrack, a source of future maintenance problems and plant stoppages. The best disposition of the intake is with the screen at right angles to the spillway so; that in flood seasons the flow entrains the debris over its crest. The intake should not be located in an area of still water, far from the spillway, because the eddy currents common in such waters will entrain and accumulate trash at the entrance. If for any reason the intake entrance should be parallel to the spillway, it is preferable to locate it close to the spillway so the operator can push the trash away to be carried away by the spillway flow. The water intake should be equipped with a trashrack to minimize the amount of debris and sediment carried by the incoming water; a settling basin where the flow velocity is reduced, to remove all particles over 0.2 mm; a sluicing system to flush the deposited silt, sand, gravel and pebbles with a minimum of water loss; and a spillway to divert the excess water. Locations for the intake and powerhouse should be selected to optimize energy generation and cost. Higher energy generation is achieved through high head and greater flow; both are considerations in selecting locations for the intake and the powerhouse. For example, if the potential intake location includes a waterfall of some meters height in its vicinity, the intake should be positioned upstream of the waterfall to exploit the additional head. If the potential location provides a tributary leading into the river, the intake structure should be positioned downstream of the tributary to exploit the additional water for energy generation. The HPP costs are strongly influenced by installed capacity, distance between the intake structure and the powerhouse, and by multiple constraints acting upon the HPP.If an HPP site offers options to position the intake and the powerhouse, an optimization analysis should be carried out that explores all possibilities, plus estimated revenues and costs, to identify the most financially attractive site. The factors to decide the position of intake structure in hydropower plant are:-Vortexes, Water resource, Debris and sediment, Available area cost and Geology condition are the main reason for allocation of intake structure position. (Canyon hydro et al, 2002)

2.7.1 Vortexes and intake position

The formation of vortex and strength are dependent on approach flow geometry, intake flow velocity, intake size and geometry, and submergence. Geometry influences are substantial and site-specific. Accurate generalized vortex prediction relationships have not been developed. Relationships which was developed through the efforts of Blaisdell (1982) and Gordon (1970), are available that indicate the potential for vortex development

$$\frac{s}{d} = c\left(\frac{v}{\sqrt{gd}}\right) = cf$$

Where S = the submergence required to prevent air-entraining vortex formation; d = the penstock diameter; V = the average penstock velocity; C = a coefficient (equal to 1.70 for symmetrical approach flow and 2.2 for lateral approach flow); and F = the Froude number of the penstock flow. If use of the above equation suggests that available submergence is in adequate or questionable, care should be taken in the design, and a physical model study is recommended. Prediction of air core vortex formation with models may be distorted by scale effects. Guidelines such as those presented by Hecker (1981) are available to assist with vortex potential evaluation. To minimize vortex potential, care should be taken to supply a well-aligned symmetrical approach flow. Ant vortex devices such as rafts (Johnson 1972), injector shafts (Bisaz et al. 1979), fixed lattice walls (Johnson 1972), etc., can be used to prevent air core formation. (Perry L. Johnson et al., 1972)

2.7.2 Water resources

As mentioned, available flow is of utmost importance for site selection. The amount of water and its annual distribution affects HPP project viability. The HPP energy output depends on annual flow distribution, hence, an HPP without a reservoir can produce energy only from available water and cannot compensate during dry periods. An HPP with a reservoir can compensate during dry periods by using water stored during wet periods and is able to produce energy during peak demand. Average river-flow increases with every tributary discharging into it hence, flow increases going downriver. Therefore, if possible at the selected site, the HPP intake should be positioned downstream from a tributary discharging into the river, which would increase energy generation. For high-head HPPs, it is common practice to increase the flow by collecting water from other springs and tributaries in the vicinity of the intake. In such cases the water of the springs/tributaries is captured and conducted to the reservoir/intake of the HPP, while strictly observing environmental standards, in particular minimum flow requirements in the tributaries. When selecting a site and estimating energy generation, it is crucial to consider hydrological flow and other factors that may reduce flow available for generation, including minimum flow (a combination of environmental and social requirements, such as ecological requirements, irrigation usage and water supply), leakage, and evaporation. (Gulliver, Rindels and Liblom, et al. 1986)

2.8 How Design and Typical Layouts

Ideally, water intakes should be located along a straight section of the stream with a stable stream bed, constant flow, bedrock and small gradient. Typically, river bends or meanders should be avoided because the inner sides of bends accumulate sediment and the outer side is subject to erosion and flood damage. If positioning a water intake on a river bend is unavoidable, the outer side is preferable because the water intake will not be subjected to blockages from sediment depositions. Intakes should be submerged deeply enough to prohibit vortex formation; typically the intake pipe should be submerged to a depth equal to three times the pipe diameter. The intake can be located within the reservoir. In that case intakes can be constructed as towers that provide water to the turbine-generator units through gated openings. The intake tower is connected to the dam with a bridge if the distance is small enough. Intake tower in reservoir created by the Boulder dam, Colorado, USA Source: Wikimedia the Tyrolean weir is a common type of structure, especially in mountainous regions. It includes the intake and is built on the riverbed itself to divert the required flow while the rest of the water continues to flow over it (Canyon hydro et al, 2002)

2.9 How the penstock length affects head pressure

The length of pipeline (also known as the penstock) has major influence on both the cost and efficiency of our system. The measurement is easy, though. Simply run a tape measure between our intake and turbine location. Computing net head is measuring head you measured gross head-the true vertical distance from intake to turbine-and the resulting pressure at the bottom when no water is flowing.net head, on the other hand, is the pressure at the bottom of your pipeline when water is actually flowing to your turbine, and will always be less than the gross head you measured due to energy losses within the pipeline. Longer pipelines and smaller diameters create greater friction.Net head is a far more useful measurement than gross head and, along with design flow, is used to determine hydro system components and power output. (Gulliver, Rindels and Liblom, et al. 1986)

Head loss refers to the loss of water power due to friction within the pipeline (also known as the penstock). Although a given pipe diameter may be sufficient to carry all the design flow , the sides, joints and bends of the pipeline create drag as the water passes by, slowing it down. The effect is the same as lowering head; there will be less water pressure at the turbine. Note that the effects of head loss cannot be measured unless the water flowing. A pressure meter at the bottom of even the smallest pipe will read full psi when the water is static in the pipe. But as the water flows, the friction within the pipe reduces the velocity of the water coming out the bottom. Greater water flow increases friction further. Large pipes create less friction, delivering more power to the turbine. But larger pipelines are also more expensive, is to size your pipe so that not more than 10%to 15% of the growth head is lost as pipeline friction. (Gulliver, Rindels and Liblom, et al. 1986)

3 Methodology

3.1 General

This study has been done with two aspects (1) evaluation of already proposed intake position (2) improvements of intake position. This means to get maximum output from any hydropower dam the intake position we put as much as possible in proper position for that reason the evaluation takes place by different evaluation mechanism. The existing intake position that proposed by METEC has its own technical advantages and disadvantages.

The data used for the study was secondary data all evaluation takes from GERD data that METEC used for optimization and after optimization. The main data used were (1) intake structure elevation (2) reservoir elevations (3) annual inflow of river (4) sediment load (5) material type of slot of gate & the main software used for analysis was MS excel which is powerful to give automatically different output for different input data insert to the software& AutoCAD to show all over structure and elevation.

3.2 The study area description

The case study project GERDP is located on the Abbay (Blue Nile) river within the Abbay River Basin in the western part of Ethiopia around 750 km by road from Addis Ababa (via Debre Marcos and Chagni). Administratively, the reservoir stretches over the three zones and ten weredas. However, all the works concerning the construction of the P_5000 scheme dam,Penstock, powerhouse, switchyard, construction camps and access roads were concentrated in An area under the jurisdiction of the SirbaAbay Wereda of Kamashi Zone of the Benshangul Gumuz Regional State.

Figure 3.1 study area location (EEPCO, 2011)

According to salini construttori s. p. a. (basic design) the basic design of GERDP has an installed capacity of 5250 MW with 15 hydro-units of 350MW. In order to improve the Hydropower projects technical and economic characteristics Ukrhydro.project Public Company carried out optimization of the HPP Basic Design in 2011, which was approved and became the basis for further designing. In accordance with GERDP project optimization the installed capacity was developed to 6000 MW with installation of 16 hydro-units of 375 MW, at which ten turbines are located on the right-bank and six on the left-bank one. And power production was increased by about 2.5% and the costs were reduced in comparison with the Basic Design. The project was optimized by METEC using different electro-mechanical (EM) equipment's technological (EEPCO et al, 2013).know the installed capacity increased to 6450MW by working different work.

Figure 3.2 GERDP 3D design photo since its design (EEPCO, 2011)

3.3 Natural Condition

3.3.1 Climatic Characteristics

Rainfall

In general, the climate of the Blue Nile River basin is characterized by season cyclist of atmospheric precipitation. In summer, moist warm masses coming from the Indian Ocean dominate over the territory (equatorial monsoons). In winter, the nature of atmospheric circulation changes; this area is influenced by Arabian dry trade winds. Wet rainfall season continues from June to October; May and October are transition periods between wet and dry seasons, while dry season is from November to May. Table 3-1 presents average long-term values of monthly and annual precipitation sums at the Rosaries weather station, which is situated near the GERDP site (EEPCO, 2013)

Table 3-1Average long-term values of monthly and annual precipitation sums (mm)

Weather station	Station elevation,(m)	Months												Year
Rosaries (Sudan)		I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	
	467	0	1	2	16	62	128	186	222	155	31	5	0	808

(EEPCO, 2013)

According to the data in table 3-2, the absolute values of daily highs of atmospheric precipitation in the examined areas occur in summer months. The number of days with precipitation of various sizes at the weather station Rosaries is presented in table 3-2

Table 3-2 daily highs of precipitation at the weather station Rosaries (mm)

Months												Year
I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	
9	15	33	45	62	77	92	93	79	44	28	0	93

(EEPCO, 2013)

In accordance with the data of the table3-3, the rainiest period of the year is summer; in the area of the planned construction, on average, every second day of summer is rainy.

Table 3-3 Number of days with precipitation of various sizes

	Months												Year
	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	
Number of days with precipitation ≥ 0.1 mm	0	0.1	0.3	2	7	13	15	16	12	4	0.7	0	70
Number of days with precipitation ≥ 1.0 mm	0	0.1	0.3	2	6	12	14	15	11	4	0.7	0	65

(EEPCO et al, 2013)

Temperature conditions: Temperature regime of the territory under consideration is -characterized by diurnal and seasonal cyclist. The western foot of the Ethiopian Highlands, where the HPP site is situated, is hotter, than the highlands themselves. Here in the afternoon the air temperature can reach 40-46°C, while during the night, even in summer, drops to 15°C.Table 3-4 shows the average air temperatures at the weather station Rosaries.

Table 3-4 Average monthly and annual - air temperatures, °C

Weather station	Station elevation,(m)	Months												Year
		I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	
Rosaries'(Sudan)	467	26.4	27.5	30	31.4	31.1	28.6	26.1	26.1	27	27	27.8	26.4	28

(EEPCO, 2013)

Data on the air average maximum and average minimum temperatures at the weather station Rosaries are presented in table 3-5.

Table 3-5 Average maximum and minimum air temperatures, °C

Months												*Year*
I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	
36.7	37.8	40.	40.6	38.9	35.0	32.2	31.7	33.3	36.1	37.2	36.7	36.7
16.1	17.2	20.0	22.2	23.3	22.2	21.1	20.6	20.6	20.0	18.3	16.1	20.0

(EEPCO, 2013)

According to the data of table 3-4 to 3-6 average annual air temperature in the area of construction is 28 °C. Average annual temperature of the warmest month (April) is 31.4 °C, while an absolute maximum is 46,0 °C. Average monthly air temperatures of the winter months is 26.4 °C, while absolute temperature minimum is 7.2 °C (January).

Table 3-6 Absolute maximum and minimum air temperature, °C

	Months												*Year*
	I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	
Number of days with precipitation ≥ 0,1 mm	43.	44	45	46	45.2	45	39.2	39	39	42	41.1	42	46
Number of days with precipitation ≥ 1,0 mm	7.2	9.4	11	14	15	16.1	18	15	16	14.5	8.5	8.9	7.2

(EEPCO, 2013)

Table 3-7 shows average daily ranges of air temperature at the weather station Rosaries. The highest average annual daily temperature range in the area of the planned construction is around 21 °C, and it occurs in winter.

Table 3-7 Average daily ranges of air temperature, °C

Months												*Year*
I	*II*	*III*	*IV*	*V*	*VI*	*VII*	*VIII*	*IX*	*X*	*XI*	*XII*	
20.6	*20.6*	*20*	*18.4*	*15.6*	*12.8*	*11.11*	*11.11*	*12.7*	*16.1*	*18.9*	*20.6*	*16.7*

(EEPCO, 2013)

Air humidity:-Characteristics of relative air humidity are presented in table 3- 8.

Table 3-8 Average monthly and annual relative air humidity, %

Weather station	Station elevation,(m)	Months												Year
Stations		I	II	III	IV	V	VI	VII	VIII	IX	X	XI	XII	
Gondar	2 270	18	21	19	27	37	59	71	75	63	42	36	30	41
Bahir-Dar	1 840	45	40	41	39	61	65	79	81	78	66	61	52	59
Asosa	1 600	28	25	38	61	73	73	74	75	73	69	51	34	56

(EEPCO, 2013)

In the mountainous, - areas average annual humidity is about 60%, the highest is 81%, the lowest - is from 18% to 39%. Closer to the construction area (weather station Asosa) average annual air humidity is 56%; the average winter (during dry season, February) - is 25%; in wet season the average monthly humidity is in the range from 70 to 75%.After the construction of the planned HPP and impoundment of the reservoir, relative air humidity in the coastal zone can come close to the values at Weather station in Bahir-Dar located on the southern shore of Lake Tana.

Wind: In this area, in summer, Indian monsoons dominate, while in winter – are Arabian trade winds. According to the seasonal nature of macro-circulation of air masses over the territory in dry season, North-Eastern and Northern winds tend to dominate, while in wet - season tend to dominate South and South-Western winds. Table 3-9 presents data on average monthly and annual wind velocities, m/s.

Table 3-9 Average long-term wind velocity at the HPP construction site is in significant (2 m/s)

Months												*Year*
I	*II*	*III*	*IV*	*V*	*VI*	*VII*	*VIII*	*IX*	*X*	*XI*	*XII*	
1.9	*1.9*	*1.8*	*1.9*	*2.3*	*2.5*	*2.2*	*2.2*	*2.0*	*1.9*	*1.6*	*1.9*	*2.0*

(EEPCO, 2013)

In winter months it is a bit lower (1.6 – 1.9 m/s), while in summer period it is a bit higher – (up to 2.5 m/s). The maximum values of wind speeds according to the project data may range from 2.5 to3.0 m/s.

Cloudiness: The value of monthly, annual cloud amounts at the weather station Rosaries are given in table 3-10. Average monthly and annual total cloudiness (amounts). According to the data in table 3-10, in mountainous and foothill areas, maximum cloudiness is in a summer wet period of the year

Table 3-10 cloudiness of area

Months												*Year*
I	*II*	*III*	*IV*	*V*	*VI*	*VII*	*VIII*	*IX*	*X*	*XI*	*XII*	
0.5	*1.0*	*1.2*	*2.0*	*3.6*	*5.0*	*5.8*	*6.0*	*5.0*	*3,0*	*1.2*	*0.7*	*3.0*

(EEPCO, 2013)

3.4 Hydrological Conditions

The Nile River basin (modern Egyptian name is el Bahr) is located in the North-East Africa. Its length is 6671 km. The basin area is 2, 870, 000 km^2. In the upper reaches the Nile takes major tributaries from the left, -El-Ghazal, from the right, Aswan, Sobat, Blue Nile and Atbara. Farther on, as a transit river, Nile flows across a tropical and subtropical semi-desert, with no tributaries along 3000 km. The Blue Nile River is Nile's right tributary most abounding in water. The river basin is located in Ethiopia and in the Northern Sudan and its length is 1600 km, Water catchment area in the mouth is 330,000 km^2, at the border between Ethiopia and Sudan is 175000 km^2, and at the HPP GERDP is 172 250 km^2.

Hydrometric studies: A list of gauging stations, on the basis of which observation data, the analysis and hydrological calculations were performed, is given in Table 3-11**.**

Table 3-11 List of gauging stations, on the basis of observation data, the analysis and hydrological calculations

Station	Location	Available observation data	Length of gauging period		
	Basin	River	Years of flow observation(n)	Period	Averaging period
2001 Kessie, F=65 784 km2	Blue Nile	Blue Nile	39	1956-2003	Annually
3025 GuderConfl., F=82 221 km2	Blue Nile	Blue Nile		1961-1968	Annually
6001 Shogole, F=156 458 km2	Blue Nile	Blue Nile	2	1959-1979	Monthly
6002 Sudan Border, F =175 000 km2	Blue Nile	Blue Nile	13	1961-2005	Annually
6000 Roseires/El Diem, F=185 000 km2	Blue Nile	Blue Nile	93	1911-2003	Every ten days

(EEPCO, 2013)

Water regime: Water regime of the Blue Nile River is characterized by the formation of a long-term flood during often rains in the rainy season. Along with ordinary rains, relatively short-term heavy showers may occur, with amount of precipitation from 100 to 150 mm. Within a significant basin area of the Blue Nile River, along with higher water content, isolated floods with duration from 1 to 2 weeks are formed. (EEPCO et al, 2013*)*

3.5 Annual flow changeability and yearly distribution

As initial data for water power calculations, the design uses a continuous flow series of the Blue Nile River (average monthly and weighted average annual water discharges, as well as monthly and annual flow volumes) for a long-term observation period from 1911 to 2005. The gauging station closest to the design site, which has data of long-term flow observations, is located 15 km downstream at the border with Sudan (6002 Sudan Border, F (field) =175,000 km^2). There are hydrometric data for this site from 1961 to 2005 with breaks in observations (totally, 254 monthly flow values you can refer from appendix 1).Given that there are virtually no lateral inflows between the site near the Sudanese border and the HPP under construction, the average monthly water discharges, obtained by the aforementioned method, were taken as inflows to the site of the HPP under construction.. The maximum inflow to the project site is 5569m^3/s in August while the minimum inflow one is 140m^3/s in April. Average annual water discharge over this period is 1559 m^3/s. Volume of annual flow varies from 20,69km^3 to 69.84 km^3, with average annual volume 49.2km^{3}Average annual flow distributions per months is presented in both discharge and volume form in table 3-12 and developed by unit hydrograph as figure 3 in discharge form and in volume form.

Table 3-12 Monthly Average flow of the Blue Nile River at the site of GERDP

Month	Flow, m^3/s	Flow volume, million m^3
January	297	795
February	195	471
March	146	392
April	140	362
May	239	641
June	674	1748
July	2647	7090
August	5569	14916
September	4677	12124
October	2463	6597
November	1030	2671
December	510	1367
Per year	1559	49173

EEPCO, 2013)

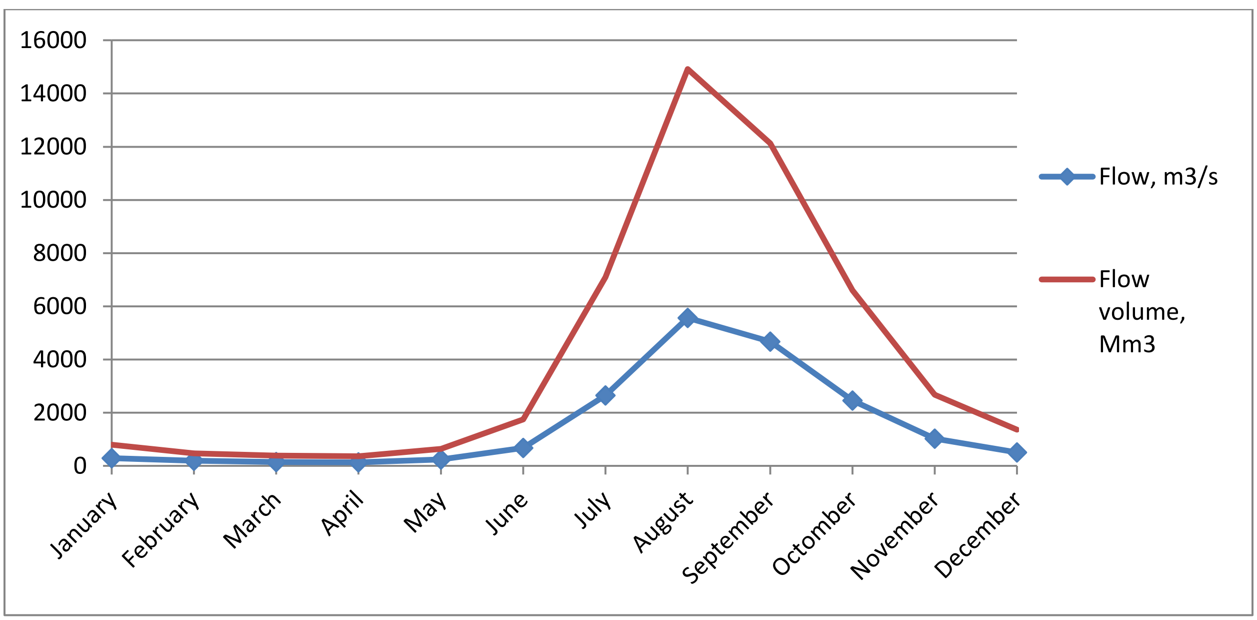

Figure 3.3 Monthly Average discharge & volume flow hydrograph of the Blue Nile River at the *HPP site*

3.6 Evaluation and improvement of intake structure position

Evaluation and improvement of intake structure: - this focus on evaluation of intake structure at different position that already proposed and comparing each other by different mechanism .to evaluate each intake structure position we will consider those factors like occurs of vortex formation, amount of water availability, sediment and debris load and cost. The proposed intake structure position has on positive and negative on over all outputs in life span of the project and based on evaluation outcomes it may needed improvement of intake position.

3.6.1 Design evaluation of hydraulic conditions of vortex formation at intake inlet

Hydropower is complicated plant. From face to tail has different main& auxiliary structures those use for power production and for other purpose. Intake structure is one of those structures that use for passage of discharge to turbine. We know the main things in hydropower production are amount of water discharges and head. Behind fixing the intake position we must cheek free from vortex formation to cheek it, first will determine all elevation position that already proposed, flow parameter, diameter of duct and dead water level after gating all parameter then will calculate submergence distance of hpp. Intake roof under upstream level should be sufficient to meet the requirement of flow entry in to water intake, i.e. to prevent air inrush into the intake through vortices (at elevation close to DWL=590.00m.s.l) and entrainment of floating objects. Vortex formation results in air influx in to waterways, which, consequently, disturbs flow continuity, reduces spillway discharge capacity, causes additional alternating loads on duct elements, and increase head loss.

In accordance with above-mentioned, it is necessary to ensure water entering in to hpp water supply duct without air influx and minimum head losses. This section presents calculated minimum admissible embedding of water intake openings under the headrace water level, which should prevent air inrush in to the intake though, air vortices. At present, in the world practice for definition of minimum admissible submergence of hpp intakes, the experts in hydraulics usually make use of empirical formulas based on field observation data and operational experience of intakes which have already been built.

In the former USSR submergence of intake opening under the minimum upstream water level was defined according to formulas based on field observation data and operational experience of 29 intakes built in Canada. It should be mentioned that four of them, vortices were observed under low water levels. The scheme of studied intakes and field observation data on conditions of frontal symmetric water supply, as well as of oblique supply in plan (or lateral one), on the basis of processed data result of mentioned field observations, the minimum admissible submergence – sill (in meter) of intake opening under which there are no vortices which entrain air in to it, can be defined with the following formulas

-for frontal symmetric water supply

$$s = 2.3v(\sqrt{d / (2g)}) \qquad 3.1$$

-for oblique water supply in plan

$$s = 3.1v(\sqrt{d / (2g)}) \qquad 3.2$$

Where d-opening height behind the pier head, m;

v-the value average velocity of steam flow in the opening, m/s

g- Gravity, m/s^2

S-submergence height

In USA and other countries intake opening submergence under the headrace level is defined on basis of chart which was also obtained on the basis of processing of a number of field observations at hpp intakes-analogues. They cover the data on 35 built and operating water intakes without vortex formation, and those 24existing with vortex formation.

-at frontal symmetric water supply

$$S = 1.7d\frac{V}{\sqrt{gd}} \qquad 3.3$$

-at oblique water supply in plan

$$S = 2.27d\frac{V}{\sqrt{gd}} \qquad 3.4$$

3.6.2 Hydraulic design evaluation reservoir volume operation option at each elevation

In hydropower amount of Reservoir volume at different elevation to be must know to determine the intake position. Reservoir volume can be categorized in to thee parts as 1, dead storage 2, life storage and 3, flood storage.

(1) Dead storage: is the volume provided to store sediments interred to the reservoir over the life span of the project. So it is designed based on the volume of sediment entered to the reservoir over the time of the project give service in which it is about 15Bm^3 as GERDP.

(2) Life storage: is the volume that covers the maximum volume of the total volume of the project and provided to store water used for energy production as discharge and/or to give head for the project and it is about 59.5Bm^3 as GERDP.

(3) Flood storage: is the volume provided to store excess water temporarily to pass it in save way without damaging the structure. And also it services as flood control from downstream of the project

The evaluation of intake structure based on ability to use reservoir water is how it use the amount of water that store in reservoir for power production. In GERD two types of intake elevation based on timing to start power generating, two early generation and fourteen normal generation the paper focus on normal generation intake position, was 560.000m now it changes to 578.850m.to evaluate on such condition. To analyze and compare the proposed intake elevations based on reservoir volume operation option was calculate all elevation vs reservoir capacity and area vs reservoir capacity graphs.

Area - capacity - Curve: The volume curve was resulted from the model done on MS excel with respect to its elevation (depth of water level) occurred within the reservoir over each 5 meter basis operation as on column 2 of the MS excel model result presented on table 16 then consider the amount of water that can be active, dead or flood to see the storage options at different intake position it takes placed with already proposed elevation. The equation that used for models to determine the reservoir capacity with the elevation is:

$$V(z) = \left(\frac{z-IL}{b_o}\right)^{\frac{1}{b_1}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.5$$

Where: z = (ma.sl) elevation of the reservoir

IL = (m.a.sl) dam site invert level

b_o= (-) coefficient = 7.282

b_1= (-) coefficient = 0.268

V (z) = (Mm^3) reservoir volume with corresponding level

The above equation is easily invertible to calculate elevation starting from volume values:

$$z(V) = IL + b_o * V^{b_1} \quad \text{……………………………………………..3.6}$$

Area - Curve: The reservoir area curve is varies with reservoir level as time function. The area obtained within the reservoir can be (1) the reservoir surface area which is varies with level and used to estimate the water lost through intake position change from the reservoir, (2) the area stretched on the body of the reservoir which is multiplied with water depth and give the volume of water in the reservoir at different and that used to calculate the water lost through the alteration of position The reservoir surface area curve was represented by the following formula (equation 3.7). The area stretched on the reservoir body was also given by the formula (equation 3.8).

The reservoir level with head and cross sectional area at every level was analyzed. So, a different number of head was occurred at different level of the reservoir and which have a crucial role on energy production. Because the reservoir is irregular shape it is difficult to calculate directly the cross sectional area at different level within the reservoir. The reservoir parameters described above were calculated and analyzed Volume-Area Curves as the following. The relationships between elevations and reservoir volumes and elevation and reservoir area are based on the Shuttle Digital Terrain Model version 4 and have been interpolated to obtain the following equations.

$$A(z) = \left(\frac{z-IL}{d_o}\right)^{\frac{1}{d_1}} \quad \text{………………….. ……………………………………………….3.7}$$

Where:

d_o= (-) coefficient = 8.54

d_1= (-) coefficient = 0.3 A = (Mm^2) reservoir surface area

$$A = \frac{V(z)}{Z} \quad \text{…………………………………………………………………………3.8}$$

Where: V (z) - reservoir volume with its corresponding level

Z - Reservoir level at different with the reservoir capacity

A - The area stretched on the reservoir body with respect its level

Reservoir head: The reservoir head can be named as gross head can gate from the area capacity curve figure and tail water level. Gross head is the elevation difference between the reservoir water level and tail water level. Reservoir water level varies with the water volume within the reservoir. And the net head is the head developed after the head loss subtracted from the gross head and that is used directly for energy production. Both gross head and net head are given as equation 3.9 & 3.10 respectively. Additionally the head loss is the head that negatively affect the energy production prom the project and it was at the following.

$H_{gross} = RWL – TWL$ ………………………………………………………………………3.9

$H_{net} = H_{gross} - H_{loss}$………………………………………………………………………………3.10

Where: RWL - reservoir supply level with respect water depth in the reservoir

TWL – tail water level

H_{gross} - gross head

H_{loss} - head loss and H_{net} - net head

Table 3-13 main reservoir parameters

Reservoir parameters	Values
1. Average annual:	
– water flow, m^3/s	1559
– annual flow volume, km^3	49.2
2. Typical design levels of reservoir (near the dam), m:	m
2.1 Normal water level (NWL);	640.0
2.2 Dead water level (DWL);	590.0
2.3 Maximum flood levels, m, for passing of:	
– probable maximum flood (PMF);	641.9
– 0.01% flood probability;	641.3
– 0.1% of flood probability;	640.8
– 1% flood probability	640.4
3. Reservoir static capacity:	km^3
– at NWL	74.5
– at DWL	15.0
– live storage between NWL and DWL	59.5
4. Reservoir surface area, km2:	Km^2
– At normal water level (NOL)	172250
– at dead water level (DWL)	600

(EEPCO et al, 2013)

3.6.2.1 Reservoir Operation model and model parameters

Here the sensitive desire was adjusting the reservoir operation schedule to release a discharge that could produce maximum energy at the possible maximum head in the reservoir. Because the annual energy production from the project is the function of the released discharge through the turbine and the net head within the storage over each operation schedule, the reservoir operation schedule plays a crucial role in getting maximum annual energy production. The optimum discharge proposed for GERDP to generate energy with vertical Francis turbine was $337m^3/s$. The turbines in GERDP are expected to operate in ±10% discharge variance without significant reduction in efficiency (EEPCO level I, volume I, 1559-1-t15, 2013). Thus first at the end of the 9^{th} month (on September 30), the reservoir volume was assumed as74.5 billion m^3 at the normal operation level (NOL) 640.23m am.sl. Then from on October 1 up to on September 30 the reservoir operation schedule was developed as column 2 - 4 of MS excel sheet resulted and presented in tabulate format on table 4-4.

Although the annual flow over the basin is constant, it is vary with the season. In rainy season the maximum inflow entered to the reservoir and in the dry season the minimum inflow entered to the storage which is even not enough for a single unit operation. In case of energy maximization the maximum discharge and head are needed at the same time. The reservoir level is the function of the inflow and outflow discharge. In order to keep both released discharge and head at balance we have to use two mechanisms: (1) adjusting the operation schedule over a short time interval, (2) as much as possible releasing maximum discharge over a maximum inflow entered to the system in order to keep the head as much as possible at average over all operation schedules with zero spill water from the system.

Based on this desire analysis have been run under different number of operation time schedules like daily, weekly, ten days, and monthly schedule.. Of these operation schedules the five days basis has been adopted. Based on the number of days within each month the last period consists 5 days for months of 30 days, 5 and 6 days for 31 days and 5 and 3days for 28 days. This is because the inflow data we have was monthly average basis, 5 days data used for the model was differ when the schedule was cross from month to month. So the interval was limited within single month. The first five interval within each month is five days but the last interval is five (5) days for the months have only 30 days, eight (3) days for only February because it has only 28 days and 6 days for the months have 31 days.

The reservoir operation model is composed of the following constraints:

1. Inflow discharge
2. Outflow discharge
3. Reservoir Area computation
4. Reservoir level computation

5. Reservoir -Area –capacity- curves
6. Evaporation losses and rainfall
7. Seepage loss through the bed of the reservoir
8. Head losses through the conveyances system
9. EM equipment main characteristics
10. Tail water rating curve.

Inflow Discharge: The input data is the annual inflow discharge to the reservoir which is found on column 3 of table 4-4 with exist number of units. Of the parameters that governed the annual energy generation, outflow (released) discharge is the main one while the released discharge again governed by the inflow discharge. Therefore the amount of annual outflow (released) discharge is depend on annual inflow discharge which was presented on table 4-5 and figure 4-4.

Outflow (released) Discharge: On column 5 the released discharge resulted on the MS excel of table 4-4 which is the model output with exist number of unit's shows the released discharge. The basic model used for reservoir operation when the water released from the reservoir was varied from the previous schedule to the next schedule was given by the main constraint of the continuity equation model to keep the annual water balance circulation as follow.

Head loss: Head loss is the head lost along the electro-mechanical structures length fittings bending and etc in which the discharge passes to the turbine. On other hand the head loss due to EM equipment main characteristics. The following types of head losses have been taken into account to calculate the net head:

- Distributed Losses along the penstocks
- Concentrated Losses at the:
    - Trash rack
    - Intake structure
    - Contractions
    - Gates

- Expansions
- Bends

According to EEPCO volume .I, 2013 of the design document, the total head losses have been implemented in the hydraulic model by the following equation:

$$\Delta H = 1.13393 * 10^{-5} * Q^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.11$$

Where: ΔH is the total head loss and Q is the discharge per unit

Evaporation loss: Evaporation loss is the loss of water from the storage by the function of the temperature of the project (GERDP) site and the surface area formed at different level of the storage with increasing up and drawdown of the storage volume. Additionally the water removed in the form of evaporation from the wet flooded area after the storage level is reduced to a lower level. Of the storage structures loss water in case of evaporation loss, reservoir and pond storage systems are the main structures directly affected by evaporation loss. The evaporation loss estimated from the HPP site was determined and presented in table 3.14.

Table 3-14 Evaporation loss

Month	Evaporation from the water surface $E_в$, mm	Precipitation P ,mm	Flow from the flooded area (fa, mm)	Additional evaporation $E_∂$,mm
January	135	0	0	135
February	136	0	0	136
March	176	7	2	171
April	168	15	5	157
May	163	81	24	106
June	127	122	37	42
July	112	165	50	-4
August	128	181	54	1
September	112	140	42	14
October	125	48	14	91
November	118	6	2	114
December	115	0	0	115
Per year	1615	765	230	1080

EEPCO level I, volume I, 2013

Turbine efficiency: it is the capability of the turbine to produce the energy with discharge passes through it and the net head we have on the project. The turbine efficiency curve could be read from the (EEPCO, 2010) report and developed.

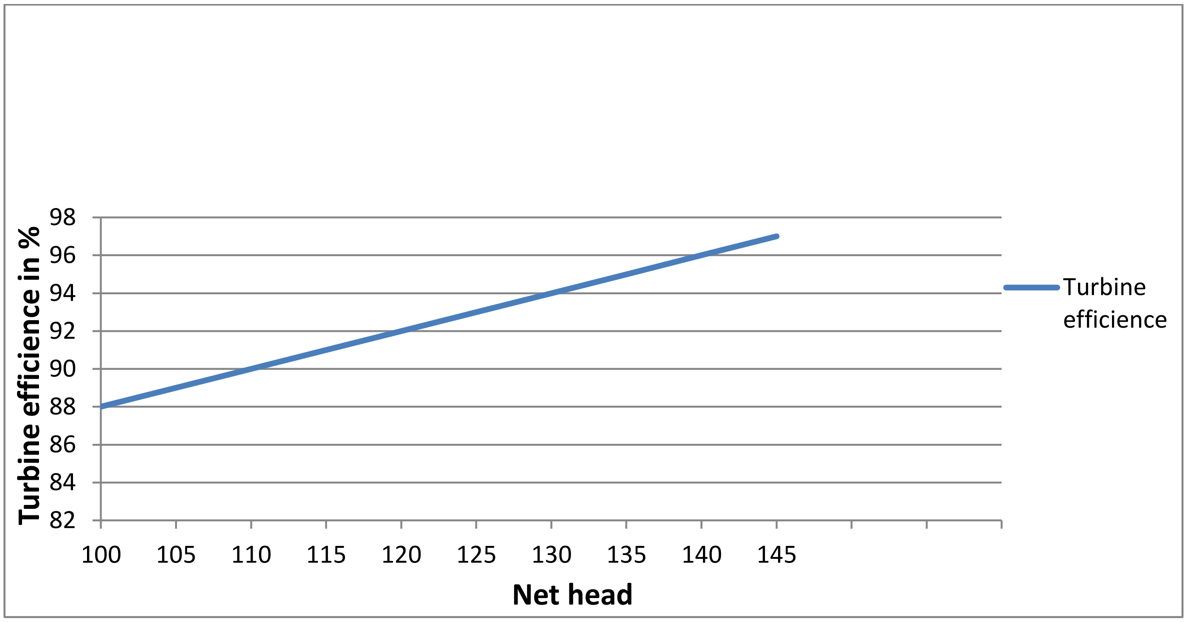

Figure 3.4 Turbine efficiency

Tail water level: Tail water level is one of the component data required on the estimation of hydropower. It was developed from the tail water rating curve of discharge vs. depth graph for different rate of plant discharge. TWL depends on the dawn stream channel geometry, discharge capacity, downstream backwater effects.

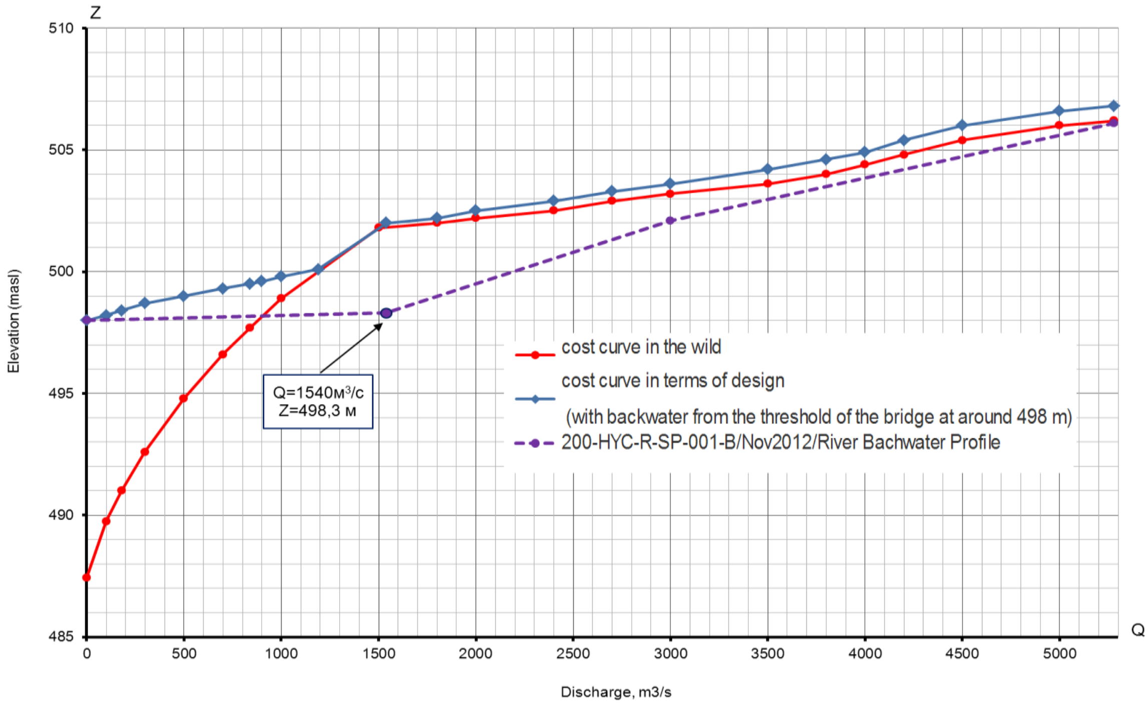

Figure 3.5 Tail water level

Downstream levels were determined by a discharge curve, shown in Fig. 3.3. This specified curve is obtained from the River Backwater Profile at different discharge released from the system

3.6.2.2 Annual Water balance

Water balance is the main important behind the reservoir operation. Conceptually it contains the meant that the total inflow and outflow over a given reservoir system should be equal. Inflow is the amount of water entered to the system over a given period of time. It may be inform of runoff, precipitation and/or discharge as form of ground water. On other hand the outflow from a given reservoir (storage) is the total amount of water removed from the system. The outflow consist the water released through the turbine in case of hydropower, water removed through the evaporation loss, and seepage loss. So the water balance over a given system is given by the continuity equation as follow:

Q_{in}-Q_{out} = 0……………………………………………………………………….3.12

Q_{in}=Inflow discharge + Precipitation ……………………………………..3.13

Q_{out} = Released discharge + Evaporation loss + Seepage loss + Spill water …3.14

Where: Qt_{in} - total inflow discharge

Qt_{out} - total outflow discharge

RO - runoff entered to the system

P - Precipitation on the system

G_{flow} - ground water flow to the system

D - Demand (released) water from the system

EVP - evaporation loss from the system

SPG - seepage from the system

3.6.3 Hydraulic design evaluation based on cost at each elevation

In any structure one criterion to say the structure design is good the structure must safe and cost-effective we know GERDP is a mega hydropower project in cost, the amount of power develop and storage capacity, when compare other hydropower structure constructed in Ethiopia it comes in first in many aspects. As we know the total cost reduced by working some modification by METEC was reduced the overall cost in 124M€. In first intake position proposed (560.000 masl) know intake position of GERD changed to new intake position (578.85 masl) it has 18.85 m difference between two elevations. All intake ducts have different gates at different position along the duct line each gates have its own embedded gate slots .to evaluate intake position in cost was calculate the cost at different intake elevation and comparing each other specially the cost of embedded part of gate slot and trash rack slot.

3.6.4 Hydraulic design evaluation based on sediment & debris at each elevation

In Hydropower scheme has influenced by different factors that affect the overall power production and lifespan of the plant main factors are sediment& debris to put intake position on ground must cheek the amount of sediment that comes in life span of the dam.

The present project layout foresees a reservoir with total capacity of 74’500 Mm^3 at Full Storage Level (FSL) of 640masl. The Minimum Operating Level (MOL) is set at elevation 590masl corresponding to a live storage capacity, between FSL and MOL of 59.5 Bm^3 The dead storage capacity beneath the MOL is of about 15Bm^3 Since the major cause of storage capacity reduction is sediment deposition, it’s important to determine the annual sediment yield rates, current location of sediment deposition, sediment densities, distribution of deposited sediment and reservoir trap efficiencies. The aim of this study is to calculate the storage loss due to the deposition of sediments, during the life time of the plant.

The different phases of sediment transportation generally occur simultaneously in natural streams, with sediment discharge classified as: suspended load and bedload. Soil erosion is a major watershed problem in the project area: eroded material derived from watershed, riverbed and banks is conveyed within the flow as sediment load, either in suspension or as bed load. Available data on the Blue Nile have shown that suspended sediments dominates the total sediment load and commonly account for approximately 90% of the total. Few measurements have indicated the relatively high sediment load in suspension during the flood, in particular clay, silt and very fine sand. The present section intends to study sediment transportation and siltation process for the GERD reservoir. We know GERD located in highly influenced by soil erodible catchment that causes the intake position of the dam highly sensitive to be cheek the sediment effect on dam intake the total amount of sediment load in life span of the dam calculated by METEC &SALINI was try to see it the influence of sediment on both intake position that proposed by METEC & SALINI.
The total sediment load as represents only a portion of the total dissolved sediment load. The unmeasured load consists of bedload plus suspended sediments in the unsampled zone between the sampler nozzle and the streambed. When the sediment sampling program is established, a preliminary appraisal should be made on the percentage that the unmeasured load is of the total load. A useful guide for evaluating the unmeasured load is the bedload correction shown in table. Five conditions are given for defining bedload dependent upon suspended sediment concentration and size analysis of streambed and suspended materials. As shown in table, either condition 1 or 2 may result in significant

bedload, which would require a special sampling program to compute the unmeasured sediment load. Conditions 3, 4, and 5 usually indicate a 2 to 15 percent correction factor, which would not require any special bedload sampling program. A special sampling program to be undertaken under conditions 1 and 2 in table 3-15.

Table 3-15 Bedload correction

Condition	Suspended sediment concentration, mg/l	Stream bed material	Texture of suspended material	Percent bedload in terms of suspended load
1	<l000	Sand	20 to 50% sand	25 to 150
2	1000 to 7500	Sand	20 to 50% sand	10 to 35
3	>7500	Sand	20 to 50% sand	5
4	Any concentration	Compacted clay gravel, cobbles, or boulders	Small amount up to 25% sand	5 to 15
5	Any concentration	Clay and silt	No sand	<2

4 Result and Discussions

4.1 General

This chapter dedicated for applying the methodology discussed in chapter 3, analyzing and discussed the result obtained from evaluation of intake structure at different elevation and comparing each result at each elevation. The results were them critically examined. The outcome the critical evaluation is expressed in next section covering evaluation by vortex formation, sediment & debris, ability of usage reservoir water and cost.

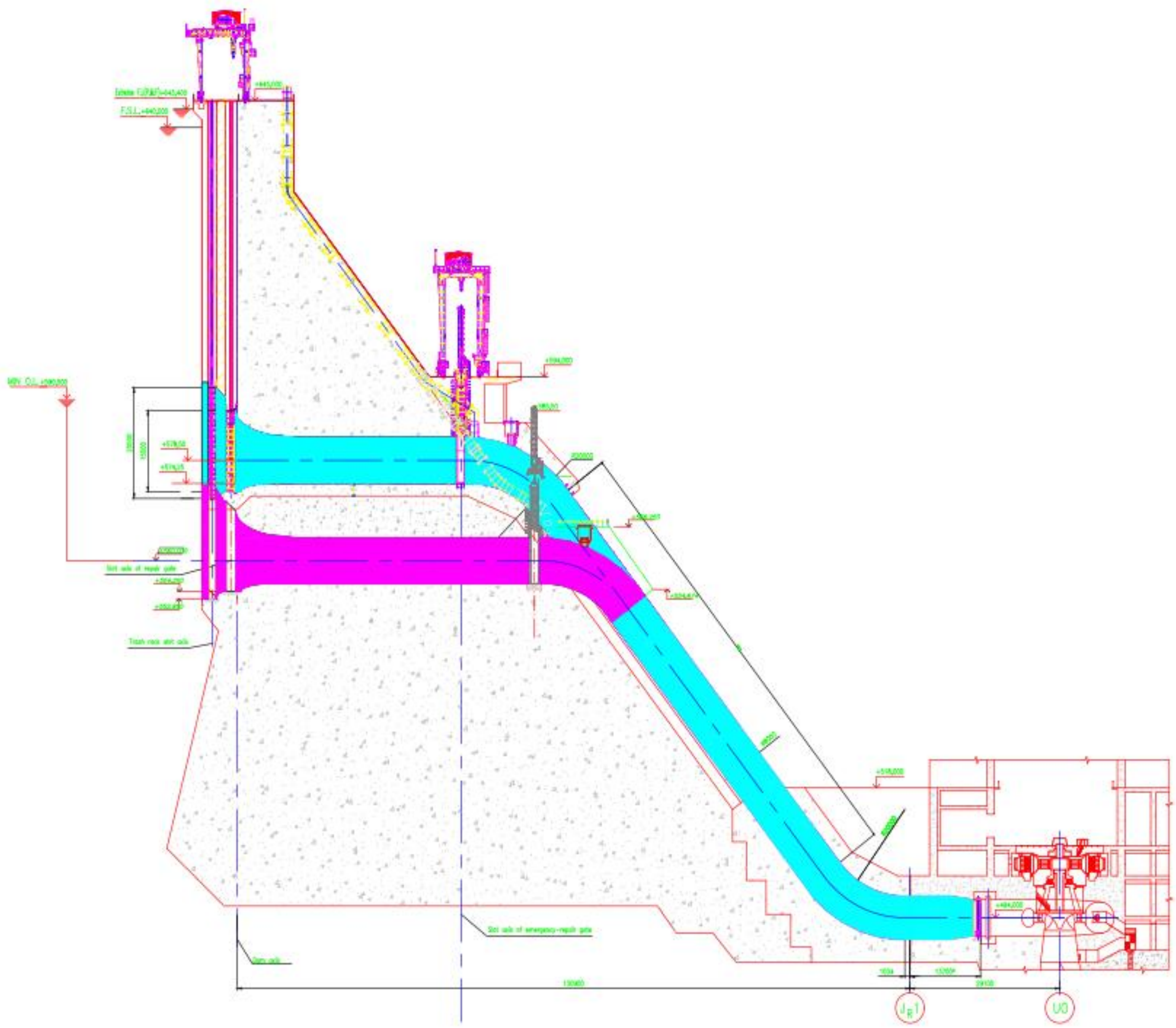

Figure 4.1 Sectional view of dam

4.2 Design evaluation of hydraulic conditions of vortex formation at intake inlet

The main purpose of design evaluation of hydraulic conditions of vortex formation at intake inlet to avoid or minimize the occurrence of vortex at intake that cause for structure deformation, decreasing of efficiency and head loss In accordance with above-mentioned, it is necessary to ensure water entering in to hpp water supply duct without air influx and minimum head losses. This section presents calculated minimum admissible embedding of water intake openings under the headrace water level, which should prevent air inrush in to the intake though, air vortices. At present, in the world practice for definition of minimum admissible submergence of hpp intakes, the experts in hydraulics usually make use of empirical formulas based on field observation data and operational experience of intakes which have already been built.

Minimum admissible submergence(S) of the intake opening roof of GERD hpp under the headrace level was defined proceeding from the conditions of prevention of air inrush and entrainment in to water supply channel through vortices, was defined with the help of formulas(3.1)-(3.4). The result of this calculation is presented in table3.1GERD hpp minimum admissible submergence values (S) for HPP intake opening roof under the headrace level-DWL 590.000m(flow 337.0unit max Q m^3/s) hydraulic conditions of flow approach to intake value of minimum admissible submergence S, m according to formulas (3.1) and(3.2) according to formulas (3.3) and(3.4)

Table 4-1 admissible submergence S, m

Hydraulic conditions of flow approach to intake	Value of minimum admissible submergence S, m	
	According to formulas (3.1)and(3.2)	According to formulas (3.3)and(3.4)
Frontal	10.0	10.46
Oblique	13.48	13.96

Table 4-2 exist admissible submergence S, m

i.no	Intake elevation	Top Sill elevation	Dead water level	Submergence, S m(DWL-sill elevation)
1	560	564.25	590	25.27
2	575.29	579.54	590	10.46
3	578.85	583.10	590	6.9
4	575.75	580	590	10

The value of intake openings submergence under headrace =DWL=+590.000masl, outer diameter is 8.5m and at intake elevation +578.850masl that is now existing in optimized design. Which S= DWL-elevation of upper roof (590.000)-(583.10) =6.9<10.0m Taking consideration the fact that frontal stream approach to the intake, so that at existing elevation the occurs of vortices high and also intake structure position was at elevation of 560.000m the submergence is,

S=DWL- elevation of upper roof (590.000)-(564.250) =25.75>13.96

Even if we taking consideration oblique stream approach that means maximum admissible distance as this elevation the occurring of vortices is low or zero.

When evaluate intake position based on formation of vortex or not the above figure described that the intake position that proposed to GERD by METEC & SALINI they have own advantage and disadvantage in occurrence of vortex the METEC proposed intake position (578.85masl) it is under vortex that means not full fill the minimum admissible submergence. In other way the SALINI proposed intake position is not affect by vortex but it admissible submergence much large than the minimum one it is unwanted length in intake not feasible.

4.3 Hydraulic design evaluation reservoir volume option at each elevation

The evaluation of intake structure based on ability to use reservoir water is the amount of water that loss or gain due to changing intake position in GERD two types of intake elevation based on timing to start power generating, two early generation and fourteen normal generation the paper focus on normal generation intake position, was 560.000m now it changes to 578.850m.when we see the relationship between amount of reservoir volume change with respect to elevation and elevation with reservoir area direct relationship more in detail in table and graph below.

Table 4-3 Elevation-area & elevation -volume calculations

ELEVATION-AREA&ELEVATION -VOLUME CALCULATIONS		
Vol.at each Elv.(Mm^3)	elevations	Area.at each Elv.(km^2)
0.862973843	500	0.592566762
6.448258374	505	2.447629758
23.65233683	510	6.120946089
61.89942153	515	12.0638615
132.9095857	520	20.67958528
250.5405341	525	32.33821713
430.6710776	530	47.38509105
691.1098359	535	66.14602278
1051.520753	540	88.93088168
1533.360556	545	116.0361604
2159.825111	550	147.7469014
2955.802613	555	184.3381874
3947.832218	560	226.0763247
5164.06705	565	273.2198025
6634.24084	570	326.0200832
8389.637587	575	384.7222644
10463.06379	580	449.5656393
12888.82291	585	520.7841771
15702.69167	590	598.6069382
18941.89814	595	683.2584365
22645.10117	600	774.9589577
26852.3712	605	873.9248409
31605.17221	610	980.3687293
36946.34471	615	1094.499794
42920.08968	620	1216.523937
49571.95333	625	1346.64397
56948.81265	630	1485.059786
65098.86171	635	1631.968504
74071.59849	640	1787.564608
83917.81246	645	1952.040076

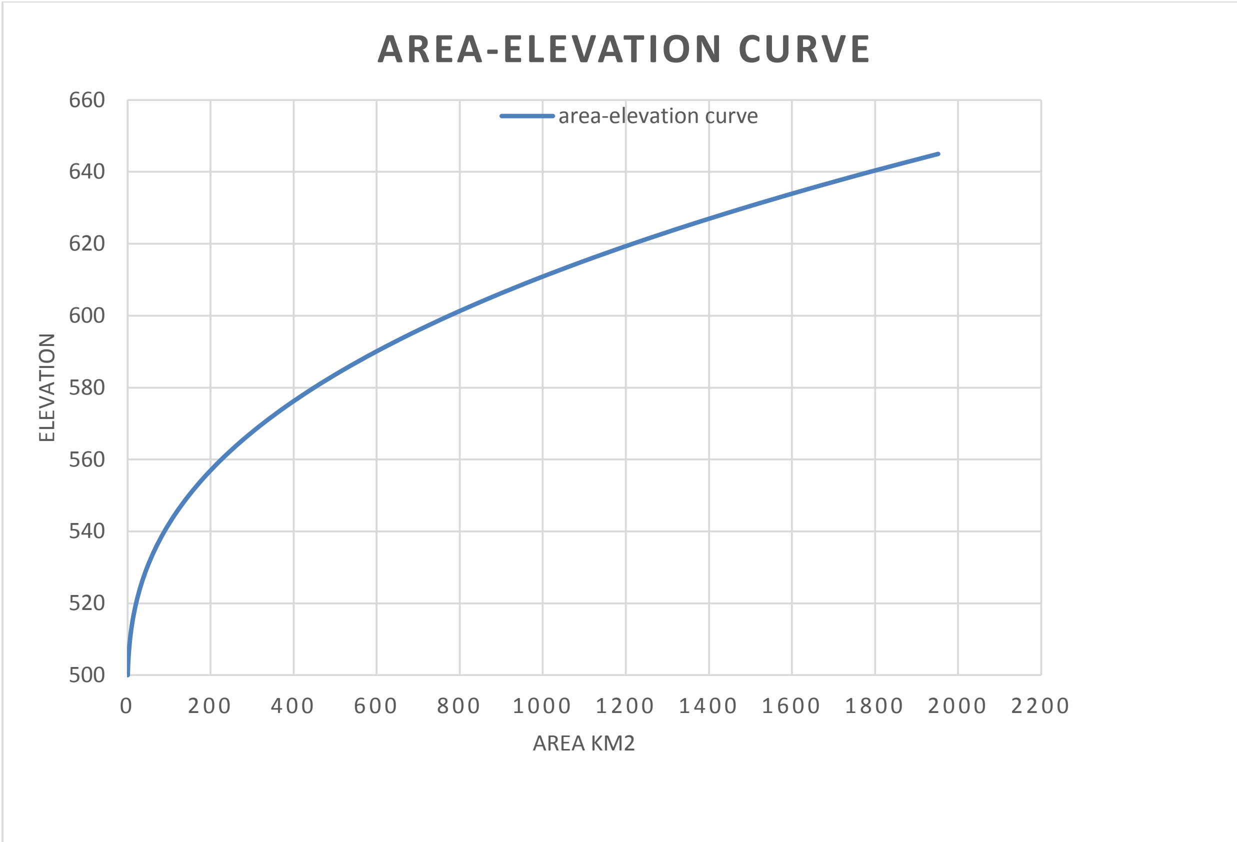

Figure 4.2 Area elevation curve

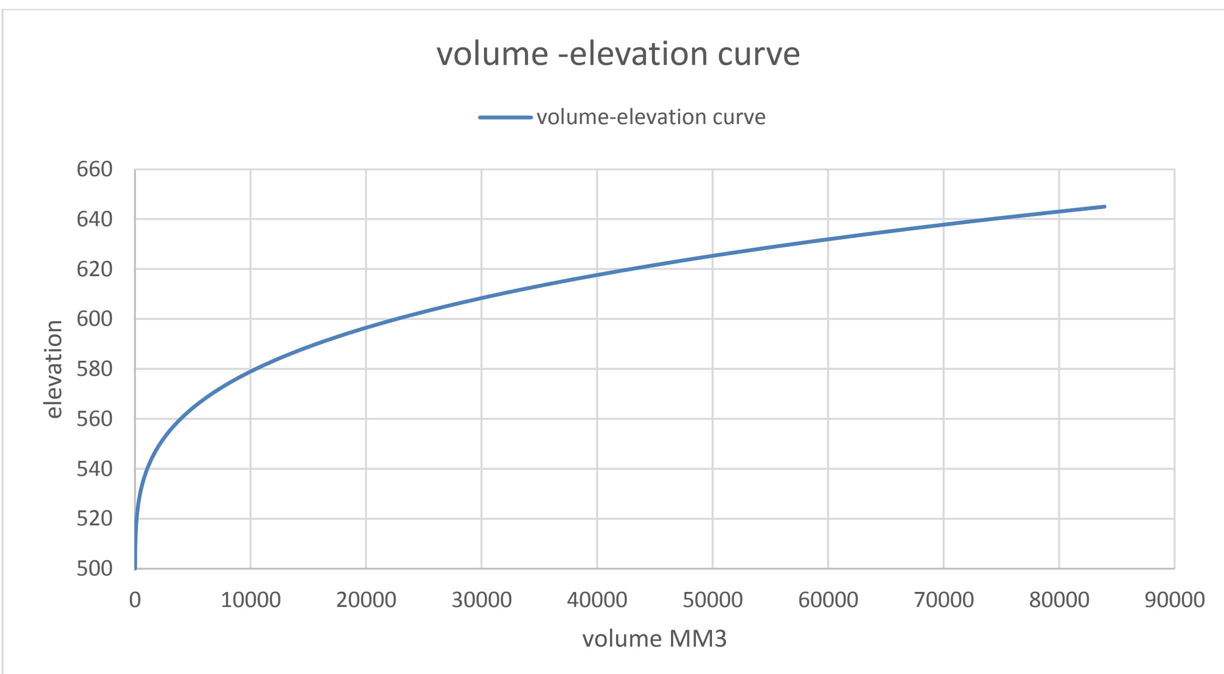

Figure 4.3 Volume-elevation curve

In the above table and two graph shows that the relation between elevation with volume and elevation with reservoir area as we know the normal water level of GERD is 640msl and the DWL is 590msl and the intake position was 560msl and now changed to 578.85msl the intake position around 18.85m has difference b/c this increment the vortex be occur to avoid the occurrence the dead water level increase about 3.56m b/c this water volume that used for power production will be decrease by around 2.2BM^3, this figure implies that the current elevation has additional water to reservoir without work and it is huge amount.

4.3.1 Reservoir operation

The numbers of the operation schedule of the modeling output over different number of operation schedule the 5days interval schedule was selected because it is more optimum than the monthly interval operation schedule. In the model the reservoir filling time was also calibrated for maximizing the annual energy production. Among different months the 9th month (September 30) is the best time whereby the reservoir is full. So on September 30 the reservoir volume is 74.5 billion m^3 at the normal water level of around 640m (a.m.sl.). From October 1 up to September 30 the reservoir operation schedule was developed and presented in tabulate. Although the seepage assume zero or neglected wasn't considered during the design of the GERDP, here it is considered.

The outflow discharge has been developed from the inflow discharge we have on the basin at the project site by adjusting the system of reservoir operation schedule with exists (16) turbine. The outflow hydrograph has been developed from outflow discharges (column 7 of table 4-4) which adjusted all over the year at 5 day's interval. The adjustment was done by meant trial and error with the objective of maximizing the energy production. The final out put deemed optimal is shown in table 4-4

Table 4-4 Reservoir operation

months	No.of periods	Inflow	Inflow	outflow	outflow	Turbine	evp loss	elv.
	in day	in m3/s	in Mm3	in m3/s	in Mm3	number	in Mm3	In m
October	5	2463	1064.016	2696	1164.672	8	26.3179	640.1601
	5	2463	1064.016	2696	1164.672	8	26.29279	640.0927
	5	2463	1064.016	2696	1164.672	8	26.26766	640.0253
	5	2463	1064.016	2359	1019.088	7	26.27888	640.0352
	5	2463	1064.016	2359	1019.088	7	26.29009	640.0451
	6	2463	1276.819	2359	1222.906	7	31.56354	640.057
November	5	1030	444.96	1011	436.752	3	34.05371	640.0432
	5	1030	444.96	1011	436.752	3	34.05636	640.0295
	5	1030	444.96	1011	436.752	3	34.05901	640.0158
	5	1030	444.96	1011	436.752	3	34.06166	640.002
	5	1030	444.96	1348	582.336	4	34.01728	639.9108
	5	1030	444.96	1348	582.336	4	33.97287	639.8194
December	5	510	220.32	1011	436.752	3	33.09633	639.6861
	5	510	220.32	1011	436.752	3	33.02792	639.5525
	5	510	220.32	1011	436.752	3	32.95946	639.4186
	5	510	220.32	1011	436.752	3	32.89093	639.2843
	5	510	220.32	1011	436.752	3	32.82235	639.1498
	6	510	264.384	1011	524.1024	3	39.28867	638.988
January	5	297	128.304	1011	436.752	3	38.31935	638.7997
	5	297	128.304	1011	436.752	3	38.20422	638.6108
	5	297	128.304	1011	436.752	3	38.08895	638.4214
	5	297	128.304	1011	436.752	3	37.97353	638.2313
	5	297	128.304	1011	436.752	3	37.85796	638.0405
	6	297	153.9648	1011	524.1024	3	45.26324	637.8109
February	5	195	84.24	1011	436.752	3	41.92115	637.5918
	5	195	84.24	1011	436.752	3	41.77319	637.372
	5	195	84.24	1011	436.752	3	41.62502	637.1513
	5	195	84.24	1011	436.752	3	41.47662	636.9298
	5	195	84.24	1011	436.752	3	41.328	636.7074
	3	195	50.544	1011	262.0512	3	24.74323	636.5736
March	5	146	63.072	1011	436.752	3	46.65465	636.3346

	5	146	63.072	1011	436.752	3	46.47501	636.0946
	5	146	63.072	1011	436.752	3	46.29507	635.8537
	5	146	63.072	1011	436.752	3	46.11484	635.6117
	5	146	63.072	1011	436.752	3	45.93432	635.3687
	6	146	75.6864	1011	524.1024	3	54.86241	635.0757
April	5	140	60.48	1011	436.752	3	43.20198	634.8303
	5	140	60.48	1011	436.752	3	43.0286	634.5838
	5	140	60.48	1011	436.752	3	42.85492	634.3362
	5	140	60.48	1011	436.752	3	42.68095	634.0875
	5	140	60.48	1011	436.752	3	42.50669	633.8378
	5	140	60.48	1011	436.752	3	42.33212	633.5869
May	5	239	103.248	1011	436.752	3	27.55771	633.3695
	5	239	103.248	1011	436.752	3	27.4563	633.1513
	5	239	103.248	1011	436.752	3	27.35473	632.9322
	5	239	103.248	1011	436.752	3	27.253	632.7122
	5	239	103.248	1011	436.752	3	27.15111	632.4913
	6	239	123.8976	1011	524.1024	3	32.43373	632.2251
June	5	674	291.168	1011	436.752	3	11.04803	632.1284
	5	674	291.168	1011	436.752	3	11.02975	632.0315
	5	674	291.168	1011	436.752	3	11.01146	631.9344
	5	674	291.168	1011	436.752	3	10.99316	631.8371
	5	674	291.168	1011	436.752	3	10.97484	631.7397
	5	674	291.168	1011	436.752	3	10.95651	631.6421
July	5	2647	1143.504	1011	436.752	3	-1.01775	632.0819
	5	2647	1143.504	1011	436.752	3	-1.02591	632.5179
	5	2647	1143.504	1011	436.752	3	-1.03405	632.9503
	5	2647	1143.504	1011	436.752	3	-1.04215	633.379
	5	2647	1143.504	1348	582.336	4	-1.04857	633.717
	6	2647	1372.205	2022	1048.205	6	-1.26272	633.9116
august	5	5569	2405.808	3370	1455.84	10	0.268654	634.4755
	5	5569	2405.808	3707	1601.424	11	0.270959	634.9482
	5	5569	2405.808	3707	1601.424	11	0.273255	635.4166
	5	5569	2405.808	4718	2038.176	14	0.274302	635.6292
	5	5569	2405.808	5392	2329.344	16	0.274519	635.6732
	6	5569	2886.97	5392	2795.213	16	0.327039	635.7259
September	5	4677	2020.464	1348	582.336	4	3.99124	636.5463

	5	4677	2020.464	1348	582.336	4	4.049227	637.3539
	5	4677	2020.464	1348	582.336	4	4.106869	638.1494
	5	4677	2020.464	1685	727.92	5	4.158388	638.8541
	5	4677	2020.464	1685	727.92	5	4.209643	639.5496
	5	4677	2020.464	1685	727.92	5	4.260637	640.2361
total	**365**		**49169.46**		**47314.8**		**1838.087**	**640.2449**

Table 4-5 Total Annual inflow and out flow

months	Inflow in Mm3	outflow in Mm3
October	6596.899	6755.098
November	2669.76	2911.68
December	1365.984	2707.862
January	795.4848	2707.862
February	471.744	2445.811
March	391.0464	2707.862
April	362.88	2620.512
may	640.1376	2707.862
June	1747.008	2620.512
July	7089.725	3377.549
August	14916.01	11821.42
September	12122.78	3930.768

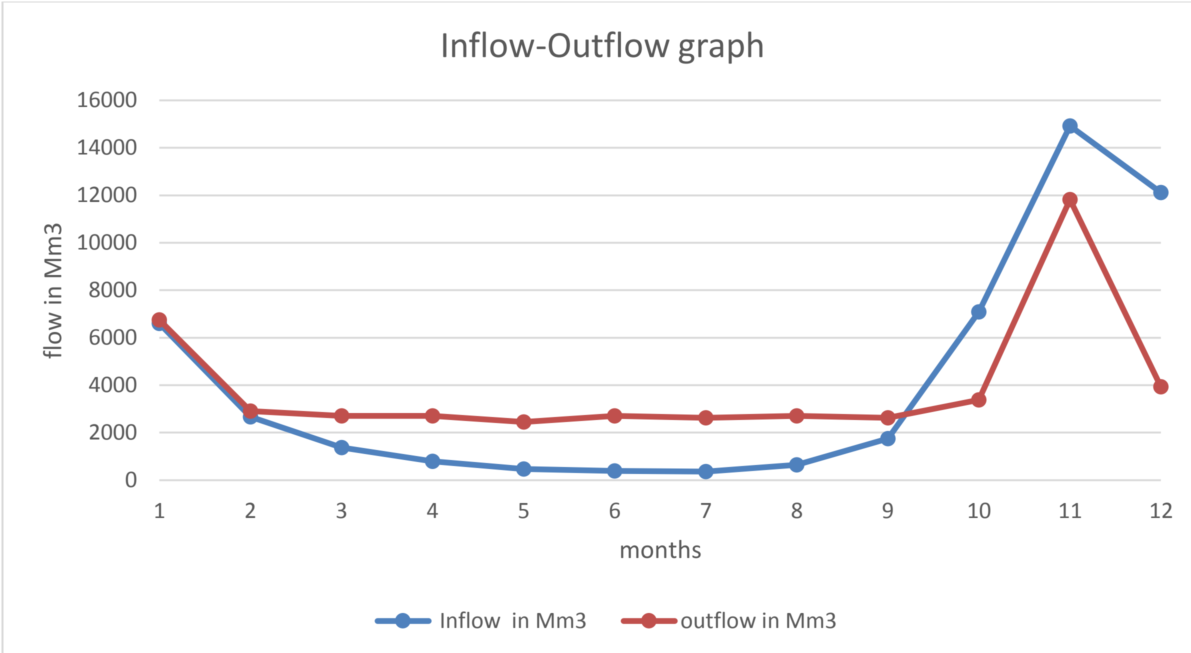

Figure 4.4 Inflow-outflow graph

4.4 Hydraulic design evaluation based on cost at each elevation

In any hydropower minimization of cost is mandatory, GERD is big hydropower plant and it needs large investment cost as we know GERD has 16 units that install to produce 6450MW. power each unit has its own gate and gate slot, the new adaptable intake position have 18.85m difference from the past position so that due to changing of position the gate slot length also shorter than the past one. In each unit it has 4 gates position and one trash rack position along the length from those gate slots 2 gate slot and one trash rack position are changed the gate slot length due to changing intake position from 560.000masl to 578.850masl.as we know to evaluate the current intake position outcomes must be check the all components of intake gate slot and trash rack slot amount and size as much as possible. when calculate the total mass that use for embedded part for slot in kilogram at each intake position by taking the total length from bottom sill up to crest and to minimize actual and calculated mass by taking factor 0.85 the amount is different we can see in table below.

Table 4-6 Gate slot mass in KG

center of elevation	Bottom sill elevation	height (m)	mass in KG per unit	(the difference mass in KG b/n 560&578.85)*0.85	total mass in KG for 14 units
				14837.5369385	207725.517139
578.85	573.85	71.15	65888.01705		
560	555	90	83343.94286		
545	540	105	97234.6		

Table 4-7 trashrack embedded part slot mass in KG

center elevation	bottom sill elevation	height in m	mass in KG per unit	(the difference mass in KG b/n 560&578.85)* 0.85	total mass difference for 14 trash rack slot
545	539.5	105.5	65203.4	9902.573236	138636.025304
560	554.5	90.5	55932.77441		
578.85	573.85	71.65	44282.68825		

As we see in above two tables the difference is shown clearly the current intake position has a great change in amount of mass (kilogram) that use for embedded part the sum of the gate slot embedded part and the trash rack embedded part around 346,361.542443KG steel and other apparatus structure. This shows that the current intake is less than the past one in total amount of mass that means decrees the total cost.

4.5 Hydraulic design evaluation based on sediment & debris at each elevation

In Hydropower scheme has influenced by different factors that affect the overall power production and lifespan of the plant main factors are sediment& debris to put intake position on ground must cheek the amount of sediment that comes in life span of the dam. In GERD located in highly influenced by soil erodible catchment that causes the intake position of the dam highly sensitive to be cheek the sediment effect on dam intake the total amount of sediment load in life span of the dam calculated by METEC. As METEC calculation the sediment volume in a year

Table 4-8 sediment distribution in pick time

Rising flood					Falling flood		
Qs(Mton/day)=0.0004286* Qw (m^3/s)					Qs (Mton/day) =1.84E-08* Qw^2 (m^3/s)		
Month	Qw	Qs	Qw	Qs	Qw	Qs	Qs tot
10 days	Jul	Jul	Aug	Aug	Sept	Sept	A+B+C
		(A)		(B)		(C)	(Mton/year)
1_10	1 452.38	6.26	5 152.19	22.08	5184.86	5.22	
11_20	2 435.65	10.51	6 119.96	26.23	4497.44	3.93	
20-31	3 889.22	18.45	5 870.99	27.68	3857.58	2.93	
	7'777.25	**35.22**	17'143.14	**75.99**	13'539.88	**12.08**	**124**

To cater for suspended sediment transported after the flood season and some sediment transported as bed load, it's recommended to increase the above value by 15%, so that:

Sediment load: 124*1.15= 143 MTons/year at Sudan Border.

From all the above it follows that the average sediment inflow volume arriving in the dam site in one year is 143 Mton, which corresponds to the 0,2 % of the reservoir total capacity.

In the GERDP it's therefore possible to use the flow duration curves for the 55 years streamflow data (the series 1950-2005) developed for each period of the flood months July-September: after the flood season, as already explained, the sediment load is almost insignificant. The relations (1) and (2) were applied to duration curves, the first to the

July and August curve, the second to the September curve, providing the average value of the seasonal suspended sediment inflow equal to 124 M tons/year:

4.5.1 Siltation

Deposited sediment volumes into a reservoir are generally computed as a multiplication of the suspended sediment load flowing through the reservoir by a certain trap efficiency. Trap efficiency of a reservoir is the ratio of the volume of sediments trapped in the reservoir to the volume of sediments entering it, in the same period of time. This ratio is necessary to estimate the time needed for a certain volume of a planned or existing reservoir to silt up.

Trap efficiency is influenced by many factors but primarily is dependent upon the sediment fall velocity, the detention-storage time, flow rate through the reservoir and reservoir operation.

There are several formulas for estimating the trap efficiency. Temporal variability of inflow, detention time of water and grain size of sediment load are the most relevant factors affecting the selection of the most appropriate formulas.as Brune and Siyam methods. Taking into account the large volume of the GERDP reservoir (74’000 Mm3) which determines a retention time of run off greater than 1 year, very high trap efficiency is expected. In the following paragraphs analytic computation is presented utilizing different approaches.

Table 4-9 trap efficiency in each 10 years

T	**Brune**	**Siyam**
years	T.E	T.E
10	97.34	99.34
20	97.3	99.32
30	97.22	99.29
40	97.07	99.19
50	96.85	99.15

Table 4-10 amount of sediment at each 10 years

T	**Brune**	**Siyam**
Years	Mm^3	Mm^3
10	1 243	1 268
20	2 485	2 536
30	3 727	3 804
40	4 966	5 071
50	**6 203**	**6 337**

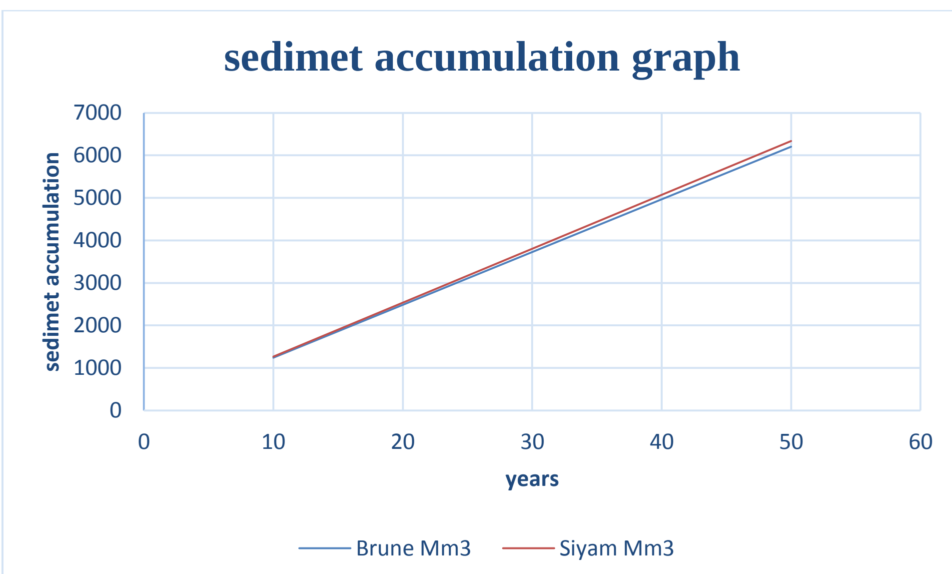

Figure 4.5 Sediment accumulation graph

Table 4-11 sediment deposition

PARAMETER	SEDIMENTATION
Specific sediment yield (ton/km^2/year)	830
Total sediment inflow (Mton/year)	143
Sediment deposition :	
After 1 year (Mm^3)	127
after 10 years(Mm^3)	1’270
After 50 year (Mm^3) NO plants u/s	6’350
After 50 year (Mm^3) with plants u/s	4’000
After 50 year of operation: Total Storage loss	8.50%

The above values are consistent with design features and power generation requirements.as we seen the amount of sediment that will be generate in catchment and silted in reservoir around 6.4BM3 after 50 years without no upstream plant this figure indicates the dead water elevation is accommodate the sediment in good manner. When we see the current intake position (578.85masl) with respect to sediment load and the past intake position (560masl) with respect to sediment load, the sediment volume it is large after 50 years it comes up to elevation 568 masl that means if we use the past intake position (+560) will be affect with sediment in other way the current intake position is clearly positioned in its place. That means the intake position not affect by sediment it have clear space b/n intake position and maximum sediment elevation. In addition to this will be upstream plant that construct in the feature that reduce the amount of sediment that come to GERD the reservoir. In other way massive work being done in abay watershed that reduce erosion in catchment. The downstream reservoir of Roseires, which is currently suffering excessive silting, will strongly benefit from reduction of solid transport due the construction of this plant. Future development of next hydropower schemes to be implemented in the upper part of the basin,

Particularly Mendaya and Karadoby, will certainly reduce the sediment transport at the GERD project reservoir. Considering the implementation of one only of the two above mentioned upstream plants, 10years after the entering into operation of GERDP, the sediment deposition in 50 years would become only 4’000MM^3 equivalent to less than 5,4 % of total storage volume.

5 Conclusions and Recommendations

5.1 Conclusions

The reviewed case study leads to some principal conclusions. These are:

i. The current intake position that already proposed by METEC that means the center of the penstock (+578.85masl) with dead water level (+590masl) is under the risk of vortex formation that affect turbine efficiency.

ii. due to increment of intake position from +560.000masl to +578.85masl vortex formation can occur, to minimize or eliminate the occurrence of vortex the dead water level increase by 3.56m because of this increment the live storage decrease around 2BM3 water.

iii. The current intake position(+578.85masl) decrease the embedded part that use for gate slot and trash rack slot by 18.85m from the past intake position (+560.00masl) due to this the current intake position is feasible than the past one.

iv. When evaluate the two intake position that already proposed based on sediment and debars the sediment accumulation after 50 years is around 6.5BM3 this can indicate the intake position +560.000 under risk because the sediment elevation goes to +568.000 and the current intake position not under risk by sediment and debars but it is much large intake position from.

v. The increment of intake position it contribute for formation of vortex and also it increase the time to fill and decrease produce power.

vi. Due to vortex formation the life span and efficiency of turbine will be decrease.

5.2 Recommendations

The study recommendations are the followings:

1. It is recommended that the current intake position reanalyze and rearrange the position to minimize vortex formation.
2. The current dead water level reanalyze and rearrange by increasing up to 4 meter to reduce the risk of vortex.
3. It is recommend that the intake position change to +575.29masl or change the dead water level to +593.46masl to avoid vortex.
4. The upstream plant must build as much as possible in feature 10years to minimize the sediment yield.
5. The upstream watershed is highly erodible area so that it needs continues rehabilitation work.
6. Regular bathymetric surveys, monitoring of sediment accumulation and reservoir Trap efficiency is recommended to assess the effects of the interventions.

6 Reference

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Appendix

Appendix A. Table: Series of hydrological data measured over abbay basin from 1911-2003

Year	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	AVG
1911	276	174	108	69	202	401	1882	5645	5320	2434	1250	612	1531
1912	321	236	134	93	93	559	2214	5302	3353	1284	625	325	1212
1913	217	145	101	100	224	154	866	2472	2350	799	285	119	653
1914	71	58	56	89	71	378	2113	6310	4398	3304	1779	650	1606
1915	325	194	127	89	209	482	1359	3058	4282	2591	1142	504	1197
1916	273	169	90	85	183	536	2782	7243	6481	4070	1736	825	2039
1917	444	277	187	143	235	679	3065	6870	8140	4070	1508	758	2198
1918	433	310	243	208	355	764	2184	4704	3345	1441	664	329	1248
1919	261	190	119	62	187	617	2688	5638	5054	1598	694	351	1455
1920	231	161	149	89	329	853	2699	4742	3823	2744	1103	493	1451
1921	291	194	116	77	161	498	1617	5600	4745	2124	845	411	1390
1922	239	157	101	69	138	563	2057	5302	4707	2557	899	455	1437
1923	265	178	153	162	403	745	2572	6720	5054	1927	918	571	1639
1924	299	223	142	204	209	706	2599	5638	5363	2214	1366	609	1631
1925	343	215	49	112	261	783	1826	4816	3819	1971	918	433	1304
1926	269	178	149	135	691	768	2733	6459	5401	2636	1030	57	1752
1927	299	182	149	93	101	714	2289	4667	3503	1949	710	370	1252
1928	213	132	93	158	575	868	3323	6571	4591	2151	988	497	1680
1929	280	194	123	143	702	1593	4107	7206	6404	3693	1246	706	2200
1930	399	256	172	204	228	621	2647	5339	4360	1598	779	403	1417
1931	235	141	97	73	82	559	1826	5712	4900	2808	992	448	1489
193	252	158	97	78	279	586	233	597	598	266	864	452	164

2							7	4	0	6			4
1933	264	173	122	96	191	467	1684	4966	5170	2662	1119	564	1456
1934	301	187	118	108	169	656	2845	6683	4900	2759	1061	594	1698
1935	328	200	135	146	409	998	4061	7049	6458	3020	1095	595	2041
1936	373	308	176	172	200	583	3145	6127	5656	2151	898	508	1691
1937	308	198	139	91	223	529	2748	6358	5054	1794	879	472	1566
1938	260	158	144	84	139	642	3406	7082	6370	3746	1249	601	1990
1939	348	224	151	140	242	627	2099	4401	4178	2423	1125	519	1373
1940	296	195	124	94	128	420	1473	5628	3954	1446	608	305	1222
1941	177	119	78	49	274	906	2303	4560	3889	2628	1187	505	1390
1942	247	153	267	111	245	549	3013	6210	4969	2568	852	454	1637
1943	285	173	108	86	156	333	1896	5536	5320	2214	928	459	1458
1944	255	166	105	96	285	610	1382	5394	4255	1563	731	390	1353
1945	230	142	84	70	282	535	2115	4740	5305	3169	1362	632	1556
1946	354	208	122	113	125	704	3541	9399	5876	2694	1169	591	2074
1947	338	214	172	265	155	402	1831	6408	5752	2457	869	515	1615
1948	274	207	163	84	130	997	2887	5419	5382	3793	1360	595	1774
1949	334	207	156	137	164	799	2886	5960	5378	2465	964	626	1673
1950	350	196	134	196	299	686	2216	5649	5347	2081	806	445	1534
1951	269	167	131	94	143	450	1697	5782	3711	2779	1217	576	1418
1952	285	176	120	94	134	442	2252	5724	4448	2432	837	414	1447
1953	230	139	105	96	212	335	2556	6596	4340	2179	897	482	1514
1954	287	176	119	97	113	630	3246	6754	5799	3200	1174	606	1850
1955	412	246	142	177	250	578	2801	6263	5887	3268	1178	613	1818
1956	340	210	143	180	164	901	2672	5614	4618	5271	1665	726	1875

1957	407	251	411	499	264	688	2114	6409	4051	1390	664	375	1460
1958	221	163	96	124	143	722	2906	7096	5320	3612	1314	633	1862
1959	393	263	170	109	228	425	2010	6041	6200	3468	1485	738	1794
1960	444	292	200	156	204	506	2629	6088	5259	2609	944	522	1655
1961	297	208	137	194	135	546	3356	6470	6474	4134	1370	831	2013
1962	428	246	183	112	209	657	2001	5625	5486	3339	987	547	1652
1963	329	198	150	160	465	585	2426	6520	5008	1813	1012	857	1627
1964	352	231	127	153	165	644	3306	6311	5621	3956	1435	729	1919
1965	429	273	167	187	104	464	1779	5006	3627	2691	1183	650	1380
1966	329	231	175	145	204	739	2346	4897	4438	1518	873	519	1368
1967	255	157	130	123	184	532	2466	5525	4977	3595	1154	750	1654
1968	328	233	126	96	110	620	3307	5986	4014	2210	772	466	1522
1969	253	193	276	143	260	678	2690	7252	4049	1492	672	359	1526
1970	217	129	121	92	97	397	2175	6430	4625	2527	945	418	1514
1971	251	147	88	62	153	649	2468	6121	4412	2085	1047	473	1496
1972	276	167	104	112	182	480	1849	3594	2643	1190	639	335	964
1973	193	111	66	65	260	671	2002	6488	4614	2405	951	464	1524
1974	290	179	140	92	291	804	2771	6411	6848	2672	1034	549	1798
1976	340	208	161	108	229	567	2067	5650	3575	1455	958	468	1315
1977	262	178	127	97	174	670	3698	5842	4553	2369	1456	556	1665
1978	294	182	132	104	187	597	2766	4676	4159	2872	938	489	1450
1979	308	206	120	94	284	641	2074	4557	3235	1617	707	371	1185
1980	216	145	100	124	169	500	2841	5591	3468	1728	689	372	1329
1981	210	128	99	96	181	411	2304	5153	4465	1953	709	361	1339
1982	228	139	120	83	119	397	1626	4076	2953	1962	696	349	1062

1983	187	126	89	95	158	420	1483	5448	3904	2122	806	377	1268
1984	206	126	71	48	109	704	2331	3228	2781	916	369	211	925
1985	119	80	56	88	273	539	2251	5708	4931	1694	663	362	1397
1986	200	128	108	114	538	546	2474	3774	3329	1402	531	274	1118
1987	155	102	131	118	300	944	1749	3755	2601	1560	738	347	1042
1988	190	149	147	84	101	809	4655	7360	5706	3741	1217	557	2060
1989	286	174	137	162	163	443	2345	4502	4110	1789	642	412	1264
1990	313	184	130	104	107	307	1742	4727	3740	1885	634	322	1183
1991	189	116	97	127	235	596	3128	5412	4268	1750	794	446	1430
1992	252	180	120	96	235	551	1567	4400	3938	2916	1249	612	1343
1993	342	208	133	268	418	1130	3135	5645	4885	2883	1280	584	1743
1994	332	197	131	106	270	741	3079	6624	5121	1657	835	413	1625
1995	191	123	100	134	200	545	1926	4840	3369	1417	622	331	1150
1996	208	129	130	186	509	1552	3877	6299	4347	2159	879	502	1732
1997	317	172	224	206	327	1047	3080	4500	2703	1828	1568	647	1385
1998	342	240	191	242	308	644	3161	7011	6011	4261	1525	729	2055
1999	431	288	169	118	321	863	3198	6285	4667	4341	1525	716	1910
2000	406	336	104	228	310	894	2897	6756	4178	3426	1523	687	1812
2001	334	188	280	308	350	1044	3576	6883	4727	2011	990	530	1768
2002	336	191	283	298	348	1045	2435	4600	3141	1372	664	404	1260
2003	260	202	223	198	158	751	3240	3981	4047	1840	658	378	1328

Appendix B. excel result